Fire Resistance of Connections in Pre-Stressed Heavy Timber Structures

By

Robert Buonomo Gerard II

Supervised By

Professor Andy Buchanan,
Associate Professor Peter Moss and
Dr. David Carradine

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Department of Civil and Natural Resource Engineering
University of Canterbury
Private Bag 4800
Christchurch, New Zealand

Abstract

Construction with composite materials has become increasingly popular in contemporary structural design for multi-storey residential, commercial, and industrial buildings. As a composite structure, pre-stressed heavy timber buildings offer sustainable, environmentally-friendly advantages over competing construction technologies utilising structural steel and concrete components. Research at the University of Canterbury is continually investigating the performance and behaviour of this composite heavy timber construction assembly. The following research report provides a fire resistance analysis for pre-stressed heavy timber structures that includes:

- A comprehensive literature review detailing the fire resistance for pre-stressed heavy timber structural components and typical connections; and
- A four-phase series of experiments with epoxy grouted steel threaded rods and proprietary mechanical fasteners to determine the fire resistance properties of steel to wood connections.

Laboratory experimentation includes cold testing to determine connection performance at ambient temperature, oven testing to evaluate heating effects on steel to wood connections, cooled testing to determine the residual strength of connections in minor fires and, finally, furnace testing to generate fire resistance design and analysis equations to be utilised for steel to wood connections.

Recommendations for the fire performance of connections in pre-stressed heavy timber structures are included in the report.

Acknowledgments

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1. Introduction

Timber is one of the most common construction materials used in structural design. The use of engineered heavy timber provides significant opportunities for advanced structural design compared to traditional light frame timber construction. Ongoing research at the University of Canterbury in Christchurch, New Zealand is studying the performance of pre-stressed heavy timber structures to determine its suitability for sustainable, environmentally-friendly residential, commercial and industrial applications.

This research project addresses the specific issues as to whether or not heavy timber construction in residential buildings presents an increased fire risk to building occupants. The literature review evaluates the fire resistance of pre-stressed heavy timber structures and experimentation determines the fire performance of steel to wood connections.

1.1. Objectives

The primary objective for the research is to determine the fire resistance of pre-stressed heavy timber buildings, focusing on structural components within the overall building system.

Secondary objectives include:

- Investigating the fire performance of steel to wood connections through experimentation at ambient temperature conditions.
- Investigating the fire performance of steel to wood connections through experimentation at elevated temperature conditions.
- Investigating the fire performance of steel to wood connections through experimentation in simulated fire conditions.
- Generating a method for calculating the fire resistance of steel to wood connections.

1.2. Background

The following survey provides general information on pre-stressed heavy timber construction and presents a background on the significance of structural fire resistance.

1.2.1. Pre-stressed Heavy Timber Construction

Advances in engineered wood technology have made it possible to construct buildings that were once limited to reinforced concrete and steel construction in timber. The Structural Timber Innovation Company (STIC, 2008) has invested significant effort to develop a building system focused on heavy timber construction. These systems utilise laminated veneer lumber (LVL) sections combined with pre-stressing steel to construct multi-storey buildings with a sustainable and environmental edge over competing construction technologies.

With any new technology, there is a natural reluctance to recommend and utilise new techniques until there is significant research and support for all relevant safety issues. Construction with timber and engineered wood products must overcome the stereotype that these buildings are more susceptible to fire. Previous research has considered the fire resistance of exposed wood members, with projects by Barber (1994), Harris (2004) and Buchanan et al (2008) evaluating the fire performance of connections in laminated veneer lumber.

Figure 1-1 – Pre-Stressed Heavy Timber Construction

While previous research has focused on specific aspects of pre-stressed heavy timber buildings in fire, a review from a holistic perspective is necessary. A full evaluation for pre-stressed heavy timber construction (as seen in Figure 1-1) is necessary to address fire safety concerns. A literature review of heavy timber and pre-stressed steel fire behaviour combined with research and experimentation with steel to wood connections seeks to evaluate the fire resistance of pre-stressed heavy timber buildings.

1.2.2. Fire Resistance

Prescribing a fire resistance is useful for quantifying the ability of a material or structural element to resist a fire. The fire resistance is the amount of time a given element can be expected to meet performance requirements when subjected to a standardised building fire. For structural systems, the fire resistance is a measure of how long a structural element can sustain a fire design load, subjected to the ISO 834 standard fire curve (FEDG, 2007). Specifying a fire resistance gives the Fire Service a conservative amount of time for which structural members can be expected to maintain stability.

A fire resistance evaluation for pre-stressed heavy timber buildings requires a thorough review of all structural components. This ranges from heavy timber beams, columns and pre-stressing steel within the structural members, to the connections within the building. The global fire resistance for the structure is the governing fire resistance for which the building is able to maintain structural stability.

1.3. Methods

There are two primary aspects for evaluating the fire resistance of pre-stressed heavy timber buildings. The first involves research into pre-stressed heavy timber components. Research focuses on a literature review compiling relevant information and determining the fire resistance of structural components. The second method seeks to evaluate the fire resistance of the steel to wood connection present in pre-stressed heavy timber buildings. This is accomplished through experimentation on two connection types; epoxy grouted steel threaded rods and proprietary mechanical fasteners.

1.3.1. Research

The first step to investigating the fire resistance of pre-stressed heavy timber buildings is to evaluate the major component parts: heavy timber, pre-stressing steel, concrete and the connections holding the building together. Studying the primary elements, heavy timber laminated veneer lumber and pre-stressing steel, dictates the overall fire performance of their combination in the form of pre-stressed heavy timber buildings. An example of a two-storey pre-stressed heavy timber model structure can be seen in Figure 1-2:

Figure 1-2 – Two-Storey Pre-Stressed Heavy Timber Model

1.3.1.1. Heavy Timber Construction

The advantages of engineered wood have made its use in construction extremely popular. Significantly stronger and straighter than sawn lumber, heavy timber, predominantly in the form of laminated veneer lumber (LVL), has increasingly been considered an environmentally-friendly alternative to steel and reinforced concrete construction. Despite the popularity of wood structures throughout the world, a stereotype exists that timber construction poses a significant fire threat to building occupants. Additional research on the fire resistance of heavy timber is necessary to evaluate this myth and draw conclusions about the fire resistance of heavy timber buildings.

1.3.1.2. Pre-Stressing Steel

The use of pre-stressing steel in construction strengthens and reinforces existing building systems. Typically used in reinforced concrete construction, the combination of pre-stressing and reinforced concrete provides significant strength and rigidity for structural systems. When evaluating the fire resistance of pre-stressed concrete buildings, the industry standard is to cite concrete's inherent fire resistance as sufficient protection for the embedded pre-stressing steel tendons. Embedment in concrete practically makes the pre-stressing steel fire resistance irrelevant. The combination of reinforced concrete and pre-stressing steel provides a considerable fire resistance due to the fire performance of concrete. The introduction of heavy timber and pre-stressing steel, however, suggests the need for further research to clarify the fire resistance of the embedded steel used in these new structures.

1.3.2. Experimentation

When evaluating pre-stressed heavy timber buildings, it is important to review all building components. The steel to wood connection in pre-stressed heavy timber buildings requires closer inspection from a fire resistance perspective. Potential design solutions include the use of an epoxy resin or proprietary mechanical fastener. Both alternatives are tested in a four-phase process to evaluate the fire resistance of the connection and evaluate steel to wood connection behaviour under fire conditions.

The experimental phases include cold testing at ambient temperature, oven testing with the connection heated to high temperatures, cooled testing where specimens are heated to high temperatures and allowed to return to ambient temperature, and furnace testing, where the connection is subjected to the ISO 834 standard fire. Results from experimentation provide data for evaluating the fire performance and predicting the fire resistance for the steel to wood connections.

1.3.2.1. Cold Testing

The first phase of experimentation is intended to replicate ambient temperature conditions. Tensile testing of the epoxies and proprietary mechanical fasteners at ambient conditions (20°C) provides ultimate load values and failure methods under typical on-site conditions. Tensile testing at ambient temperature also determines the ultimate load for the connection. This allows for a comparison with behaviour at elevated temperatures, providing valuable information regarding connection performance and effects at high temperatures.

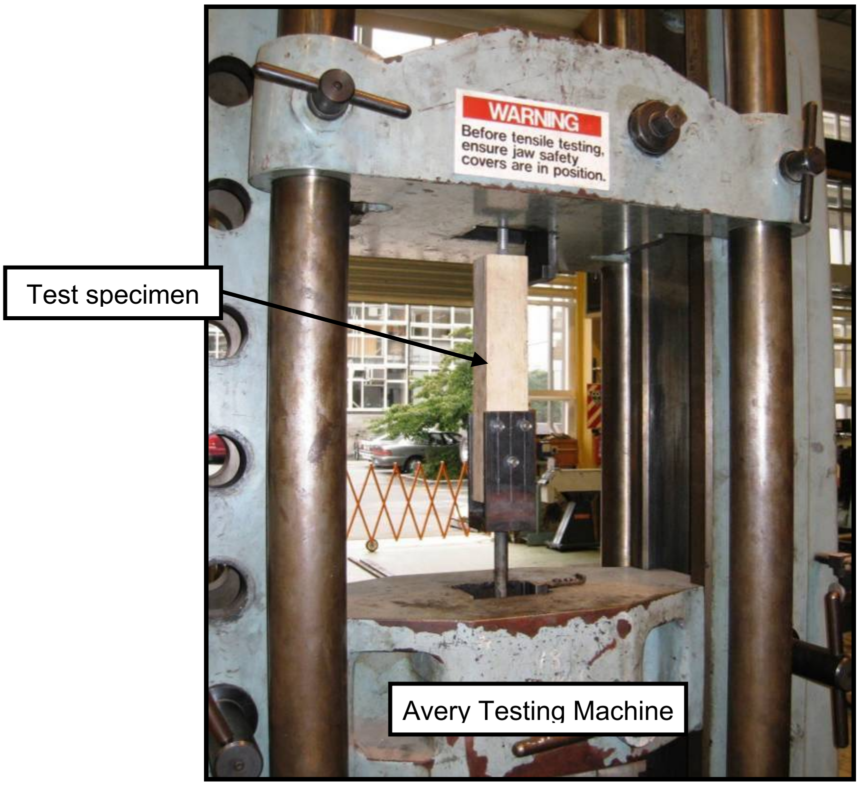

Figure 1-3 – Cold Testing Apparatus

Ultimate load values from ambient testing are to be used in subsequent phases of testing. The ultimate tensile load provides a benchmark for which each specimen can be expected to perform under normal conditions. An example of the cold testing set up is shown in Figure 1-3.

1.3.2.2. Oven Testing

Epoxy use as an effective connection material at ambient conditions is widely accepted. Previous experimentation has shown that epoxy adhesive loses strength at elevated temperatures, with the maximum recommended strength for use limited to 100°C (Barber, 1994). Over time, manufacturers have released high temperature epoxy. This product is intended to display improved performance at high temperatures.

The second phase of testing is intended to evaluate the strength and performance of high temperature epoxy at elevated temperatures. Steel to wood connections are heated to temperatures ranging from 50°C to 200°C with all specimens subjected to tensile testing to determine the ultimate load. Results from oven testing can be used for comparison with cold test results to evaluate connection behaviour and performance at elevated temperatures.

1.3.2.3. Cooled Testing

In minor fires, heavy timber components are exposed to high temperatures without experiencing significant charring or pyrolysis. Cooled testing of epoxy grouted test specimens seeks to determine the post-fire behaviour and strength of steel to wood connections. This is intended to simulate a minor fire. Epoxy specimens are heated similar to oven testing, but allowed to return to ambient conditions prior to tensile testing.

Ultimate strength comparisons between test scenarios can be used to identify the effects of heating and cooling on epoxy adhesives. Results from all testing phases, cold, oven, and cooled tests, can be compared to determine steel to wood connection strength and behaviour.

1.3.2.4. Furnace Testing

The final phase of experimentation, furnace testing in the custom furnace, is intended to determine the fire resistance of steel to wood connections. Specimens are subjected to constant tensile fire load, a fraction of the ultimate load under cold conditions, and exposed to the ISO 834 standard fire in the custom furnace. Previous investigations (Harris, 2004 and Lane, 2001) provide methods that are used for calculating the fire resistance for steel to wood connections in pre-stressed heavy timber buildings. Test specimen pyrolysis is shown in Figure 1-4.

Figure 1-4 – Furnace Testing Apparatus

1.4. Organization of Thesis

This report consists of 12 chapters, including an appendix.

- **Chapter 2** provides an overview of pre-stressed heavy timber buildings.
- **Chapter 3** discusses the fire resistance of pre-stressed heavy timber structural components.
- **Chapter 4** summarizes the materials and testing procedure associated with experimentation of steel to wood connections.
- **Chapter 5** includes cold testing experimentation and cold test results.
- **Chapter 6** provides oven testing experimentation and oven test results.
- **Chapter 7** presents cooled testing of steel to wood connections and cooled test results.
- **Chapter 8** presents furnace testing, the fire resistance calculation for test specimens and a design and analysis method for determining the fire resistance of steel to wood connections.
- **Chapter 9** provides a discussion of experimental issues.

- **Chapter 10** provides conclusions and recommendations for the research.
- **Chapter 11** includes the references for the paper.
- **Chapter 12** includes the Appendix.

2. Pre-Stressed Heavy Timber Structures

Prior to experimentation and analysis of connections, a survey of pre-stressed heavy timber structures is necessary. A review of the behaviour and structural composition of pre-stressed heavy timber buildings is intended to enhance the understanding of this innovative construction type.

2.1. Pre-Stressed Heavy Timber

Timber is one of the most readily available and popular construction materials in use today. Significant advances in timber construction technology have led to the development of pre-stressed heavy timber construction. This allows for the construction of multi-storey buildings to be built to greater heights and longer spans than ever before.

2.1.1. Design Objectives

The introduction of pre-stressed heavy timber construction provides an innovative structural assembly that offers many competitive advantages over existing construction methods. Composite behaviour of heavy timber and pre-stressing steel allows for the construction of sustainable, environmentally-friendly, multi-storey buildings for residential, commercial and industrial occupancies. The Structural Timber Innovation Company (STIC), a leader in pre-stressed heavy timber design, demonstrates the advantages of pre-stressed heavy timber compared to traditional assemblies claiming these new structures are (STIC, 2008):

- Less expensive to construct through prefabrication and comprehensive design.
- More resistant to seismic events and other natural disasters.
- Less energy-intensive in construction materials and life-time use.
- Less wasteful of materials with lower environmental emissions.
- Safer in fire events and other hazardous situations.

As this sustainable construction assembly gains acceptance, the application of pre-stressed heavy timber continues to grow. Multi-storey structures up to three stories have been constructed in Australia, with many three to four storey structures under construction in New Zealand and even six to eight storey structures in Europe and the United States (Rowbotham, 2009).

2.1.2. Design Methodology

The pre-stressed heavy timber assembly consists of a heavy timber frame with pre-stressing steel and a concrete composite timber deck. Gravity resisting elements including heavy timber beams, columns and walls, and are composed of laminated veneer lumber (LVL) or glulam timber. Pre-stressing steel embedded in the engineered wood beams and walls provides increased strength and stiffness, achieving longer spans and greater heights compared to typical heavy timber buildings.

The combination of pre-stressing steel with heavy timber construction as a “hybrid” system was first proposed by the Precast Seismic Structural Systems (PRESSS) Research Program (Priestley, 1991). This assembly, commonly referred to as a jointed ductile connection, includes post-tensioned, unbonded steel tendons within the heavy timber beams and walls. Composite behaviour increases both the structural strength and stiffness required to construct large, multi-storey structures (Buchanan et al, 2008).

The gravity load resisting system also includes a timber-concrete composite floor system. The floor system, adapted from reinforced concrete construction, utilises LVL beams supporting plywood that acts as formwork for a reinforced concrete slab that provides diaphragm support and rigidity (Yeoh, 2008).

The lateral load resisting system for pre-stressed heavy timber buildings is composed of multi-storey frames and pre-stressed heavy timber walls (Smith et al, 2008a). Moment resisting frames using unbonded steel in heavy timber structural elements are used in conjunction with pre-stressed heavy timber shear walls to provide lateral resistance to wind or seismic loadings.

2.1.3. Design Features

While pre-stressed heavy timber structures present many advantageous design features, some of the predominant aspects include environmental considerations, prefabrication of structural systems and potential cost savings.

Composed of engineered wood products including laminated veneer lumber and glulam wood, heavy timber construction is the only renewable and sustainable construction method currently in use (Rowbotham, 2009). Further, the use of heavy timber results in less energy-intensive structures, with lower CO_2 emissions and represents a significant advance in meeting carbon neutrality objectives (STIC, 2008). Using engineered wood products as opposed to concrete or structural steel reduces material waste and cuts down on environmental emissions during the construction and re-use processes.

Prefabrication of the timber-concrete composite floor system presents a significant advantage for pre-stressed heavy timber structures. Heavy timber members can be constructed off-site and assembled in place, with a monolithically poured slab connecting the panels across the floor diaphragm. This prefabrication process allows for easier transportation and installation of floor panels, better in-plane strength and stiffness resulting from a monolithically poured slab, as well as considerably faster construction (Buchanan et al, 2008).

A preliminary cost estimate compared the cost of constructing a case study building using pre-stressed heavy timber compared to steel and concrete construction (Smith et al, 2008a). Despite the timber alternative costing slightly more than either the steel or concrete options, the analysis suggested that cost-savings might be more comparable if further analysis considered additional details including the cost of construction time saved from the use of prefabricated materials, material savings costs for wood compared to concrete and steel, as well as sustainability, energy and carbon emission considerations to the environment.

2.2. Current Projects

Progress with pre-stressed heavy timber construction has been ongoing on two fronts; development of timber-concrete composite structures and research into pre-stressed heavy timber structural behaviour. As research progresses and understanding of the benefits of sustainable construction advances, timber-steel composite construction has grown in popularity.

Perceiving the need for additional understanding of performance and behaviour of pre-stressed timber structures, academic research establishments including the University of Auckland and University of Canterbury in New Zealand are engaged in on-going research and experimentation with pre-stressed heavy timber structures.

2.2.1. Multi-storey Hybrid Wood-Steel Construction

The application of a composite wood-steel structural assembly has gained popularity in construction practice. An increasing number of projects are taking advantage of the combination of the strength inherent in steel framing as well as the familiarity and cost savings associated with engineered wood construction. Examples of timber construction with a structural steel frame are shown in Figure 2-1 and Figure 2-2:

Figure 2-1 – Six Storey Timber-Steel Composite Construction, La Jolla, California, USA (Cheung, 2008)

Sawn lumber and engineered wood products are among the most popular construction materials for residential homes in the United States (Cheung, 2008). While maintaining the familiarity of wood construction, the addition of steel elements to the construction assembly strengthens the structural systems, allowing for the construction of larger and more extravagant buildings.

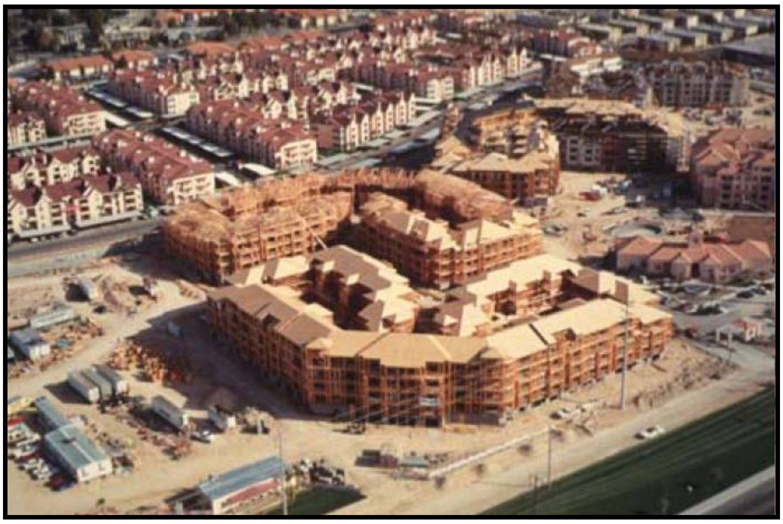

Figure 2-2 – Five Storey Timber-Steel Composite Construction, Las Vegas, Nevada, USA (Cheung, 2008)

2.2.2. Pre-Stressed Heavy Timber Research

In light of the growing popularity, additional research is required to more fully understand the potential advantages associated with pre-stressed heavy timber construction. On-going research at the University of Canterbury has been engaging in feasibility and performance investigations to gain further insight into the benefits of pre-stressed heavy timber construction.

2.2.2.1. Feasibility of Pre-Stressed Heavy Timber Buildings

Prior to investigating the behaviour of pre-stressed heavy timber structures, feasibility studies analyzed and compared design methods and construction options for pre-stressed heavy timber compared to alternative construction techniques. Research by Smith (2008b) for a six storey case study building evaluated the structural design, analysis and construction methods for pre-stressed heavy timber compared to reinforced concrete and structural steel construction. Additional research included a comprehensive study of the life-cycle energy use and potential energy savings for pre-stressed heavy timber construction compared to alternative construction materials (Perez, 2008).

Following any contemporary development in construction method, building codes seek to remain progressive and "catch up" with the new building techniques. The same is true for the use of timber-steel composite construction. A building code analysis performed by Cheung (2008) evaluated changes related to timber-steel construction in the 2006 International Building Code, the predominant building code in the United States. Increased understanding of these unique structures has encouraged revisions to the code allowing greater height and floor area allowances in addition to more flexible fire resistance requirements (Cheung, 2008).

2.2.2.2. Seismic Performance of Pre-Stressed Heavy Timber Buildings

A primary objective of the Structural Timber Innovation Company (STIC) is to enhance the understanding of the seismic performance of pre-stressed heavy timber buildings. Significant research has investigated the performance of multi-storey buildings using pre-stressing (Newcombe, 2008) and laminated veneer lumber (Palermo, 2005). In depth analysis of pre-stressed timber columns and beams was performed by Iqbal (2008). Testing with the steel-heavy timber floor system was engaged by Yeoh (2008) and O'Neill (2009). Increased experimental testing has served to promote the understanding of pre-stressed heavy timber construction, with significant opportunities for future research.

2.2.2.3. Details of Pre-Stressed Heavy Timber Buildings

A structural element analysis provides a significant advance in pre-stressed heavy timber understanding, however it is equally important to research the connections holding the building together. A survey of the feasibility of connection types was performed by Smith (2008b) and Halliday (1991). Specific experimentation researched the column to foundation joint (Pasticier, 2006) and the beam to column joint (Iqbal, 2009). A timber frame analysis including connections was performed by Batchelar (2006) with additional joint testing by Gaunt (2001).

While numerous research projects have served to enhance the understanding of connections and detailing with pre-stressed heavy timber buildings, one connection in particular requires additional attention. A significant part of the seismic energy dampening system, seismic dissipaters in the form of epoxy grouted steel rods present a concern when faced with a fire scenario. Previous research performed by Barber (1994), Buchanan (1996, 1999), Deng (1997), Korin (1997), and Harris (2004) has investigated the fire performance of epoxy grouted steel rods in glulam and LVL. The recent introduction of high temperature epoxy, in place of all-purpose epoxy, has suggested further research may be necessary for increased understanding of this unique connection.

2.3. Structural Composition

Traditional construction relies on a primary construction material for gravity and lateral resistance, such as composite steel-concrete construction. Pre-stressed heavy timber structures, however, utilise a range of construction materials including heavy timber, pre-stressing steel and reinforced concrete.

2.3.1. Heavy Timber

Conventional timber framing is one of the most common residential construction materials in practice. While traditional methods use "stick-framing" and light frame timber construction, engineered wood production has made heavy timber construction considerably more popular.

Heavy timber construction is composed of one of two types of timber. These include traditional wood with sawn lumber and engineered wood with glulam or laminated veneer lumber. The high cost of natural lumber as well as the challenge in finding large, uncut sections makes finding heavy timber structures built from sawn lumber increasingly difficult. Engineered wood products consisting of glulam or LVL (as seen in Figure 2-3), however, have gained popularity in many residential and commercial applications because of the increased strength and availability compared to sawn lumber.

Figure 2-3 – Heavy Timber Construction with Glulam Framing

Both the glulam and LVL engineered wood products provide increased structural strength when compared to typical sawn lumber. Strips from natural trees are cut to thin veneers starting at 3mm wide, oriented perpendicular to each other, and glued and pressed firmly together to create large cross sections of wood. These products can be made to any dimension and provide significant strength, as they are free of knots and inconsistencies found in typical sawn lumber. Improved strength and performance at competitive cost has made engineered wood products an increasingly popular construction material.

2.3.2. Pre-Stressing Steel

Typically used in reinforced concrete construction, the addition of pre-stressing steel to existing building systems strengthens the structural system making it capable of spanning longer distances and building to greater heights. Pre-stressing steel can be pre-tensioned or post-tensioned to force a structural element into compression. Pre-stressing steel forces structural members into compression, providing increased strength and resistance for building systems (Pampanin, 2005) and is typically used in concrete construction due to concrete's high compressive strength and weak tensile strength.

Pre-stressing steel in building applications requires the addition of two structural components. High strength steel tendons span the length of a building element and are held in place by large plate anchors at either end. Tendons within the beam are held by the large plate anchor maintaining the compressive force on the beam, as shown in Figure 2-4:

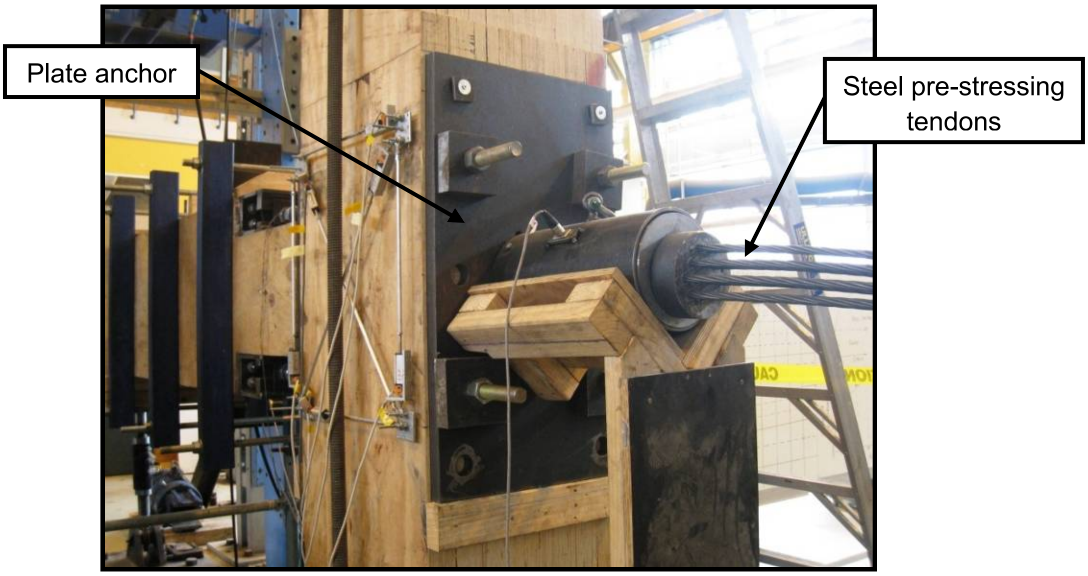

Figure 2-4 – Steel Tendons and Plate Anchor for Post-Tensioned Beam-Column Connection

2.3.3. Timber-Concrete Composite

A primary feature of the pre-stressed heavy timber building is the composite timber-concrete floor system. Prefabricated timber panels can be constructed off-site and installed in the pre-stressed heavy timber building prior to pouring of the concrete floor slab. A monolithically poured floor slab provides significant stiffness and rigidity to the floor diaphragm in addition to considerable acoustic performance between the floors (Yeoh, 2008). A notched connection in the heavy timber beams provides a positive connection for composite action between the two construction materials. A timber-concrete composite floor used in the case study building analyzed by Smith (2008b) is shown in Figure 2-5:

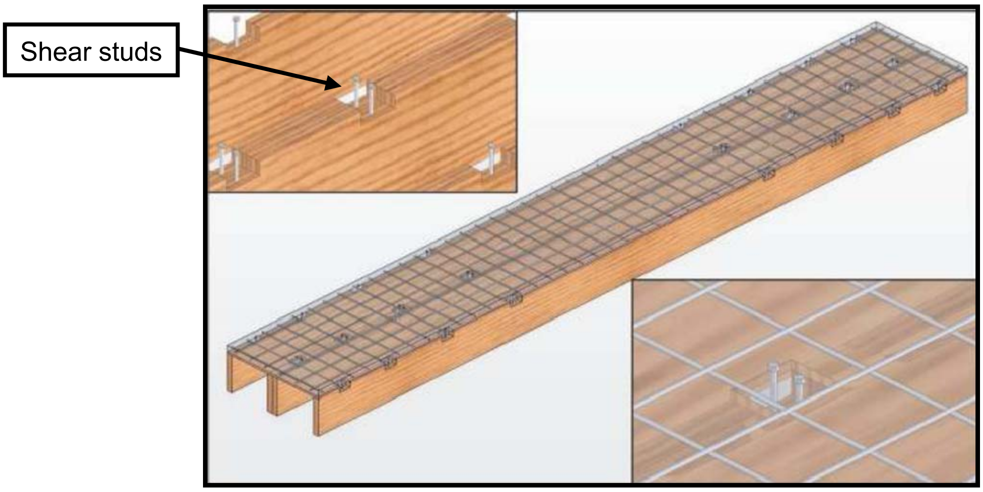

Figure 2-5 – Timber-Concrete Composite (Smith, 2008b)

2.3.4. Steel Energy Dissipaters

Pre-stressed heavy timber structures utilise a hybrid system introduced by the University of California, San Diego (Priestley, 1991). Sacrificial mild steel energy dissipaters located at beam, column and wall joints absorb lateral load during a seismic event. Typically used in reinforced concrete applications, the adaptation to heavy timber structures has provided the means for focusing lateral loads into a replaceable connection within a connection joint, sparing structural elements from seismic damage. A typical energy dissipater is shown in Figure 2-6:

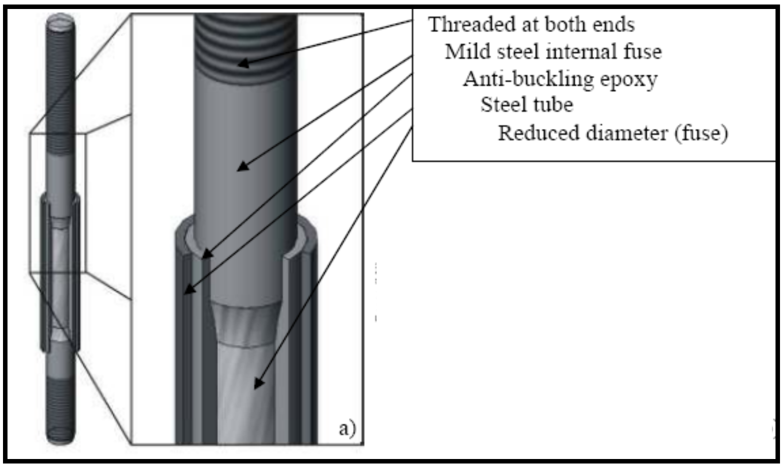

Figure 2-6 – Energy Dissipaters (Smith, 2008b)

2.3.5. Connections

Designing and detailing connections for pre-stressed heavy timber requires the use of many structural connection tools. High strength steel plates are used to maintain the pre-stressing in the steel tendons within timber beams and walls (as seen in Figure 2-4). Wood screws are used for joist hangers at the joist to beam connection as well as for corbels to support the beam to column connection. A screw application for the timber corbel can be seen in Figure 2-7:

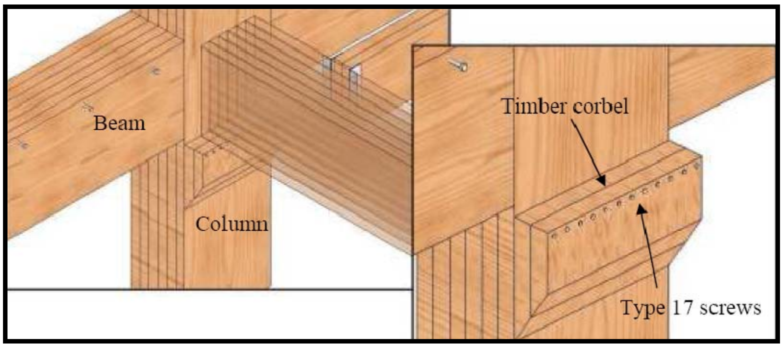

Figure 2-7 – Column to Beam Connection (Smith, 2008b)

High strength bolts are used for the beam to column connection in addition to the column to foundation and wall to foundation connections (Iqbal, 2009). Typical foundation connections are designed through the use of embedded steel rods in concrete. As a steel product embedded into wood, the steel to wood connection requires a creative solution for embedment in laminated veneer lumber as opposed to concrete. Epoxy adhesive is used to create an epoxy grouted steel rod connection for steel to wood connections in pre-stressed heavy timber buildings (Smith et al, 2008a). Performing well under ambient temperature conditions, the epoxy grout provides a strong connection between the laminated veneer lumber and steel threaded rod.

2.4. Structural Components

The pre-stressed heavy timber building combines a variety of construction components for building design. Heavy timber beams, columns and walls resist gravity loads. Pre-stressing steel provides additional strength in beam and wall elements. A timber-concrete composite flooring system provides a prefabricated solution to floor installation, acting as a rigid diaphragm for the transfer of gravity and lateral forces. Structural elements are connected using wood screws, steel bolts and plates, and epoxy grouted steel threaded rods. A basic layout of the primary structural systems in pre-stressed heavy timber construction is shown in Figure 2-8:

Figure 2-8 – Two Storey Pre-Stressed Heavy Timber Model

2.4.1. Typical Beams

One of the dominant features of pre-stressed heavy timber construction is the use of pre-stressed heavy timber beams located throughout the interior and exterior of the building. Composed of hollow laminated veneer lumber sections with pre-stressing steel embedded in the centre, these structural elements are the main gravity load resisting elements for consolidating load to the heavy timber columns. Heavy timber beams can be seen in Figure 2-9:

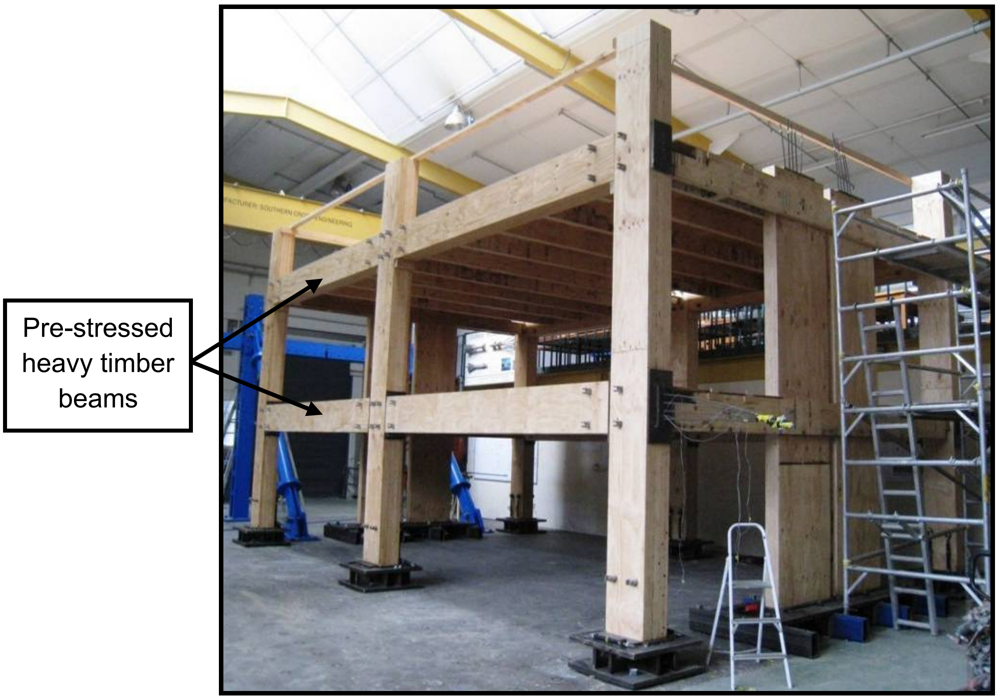

Figure 2-9 – Typical Pre-Stressed Heavy Timber Beams

Typical beams in pre-stressed heavy timber structures are equipped with energy dissipaters in the form of mild steel rods epoxy grouted into the beam and inserted into the column. Energy dissipaters are intended for use in seismic events only, and absorb the lateral load to prevent damage to the structure. For more information on heavy timber beams refer to Iqbal (2008).

2.4.2. Typical Columns

Typical columns used in pre-stressed heavy timber construction are composed of laminated veneer lumber sections, continuous throughout the height of the building. Typical columns carry gravity load to the foundation of the structure. Unlike heavy timber beams, columns are solid in section and do not possess any pre-stressing steel. A typical interior column is shown at centre, with a typical exterior column shown at right in Figure 2-10:

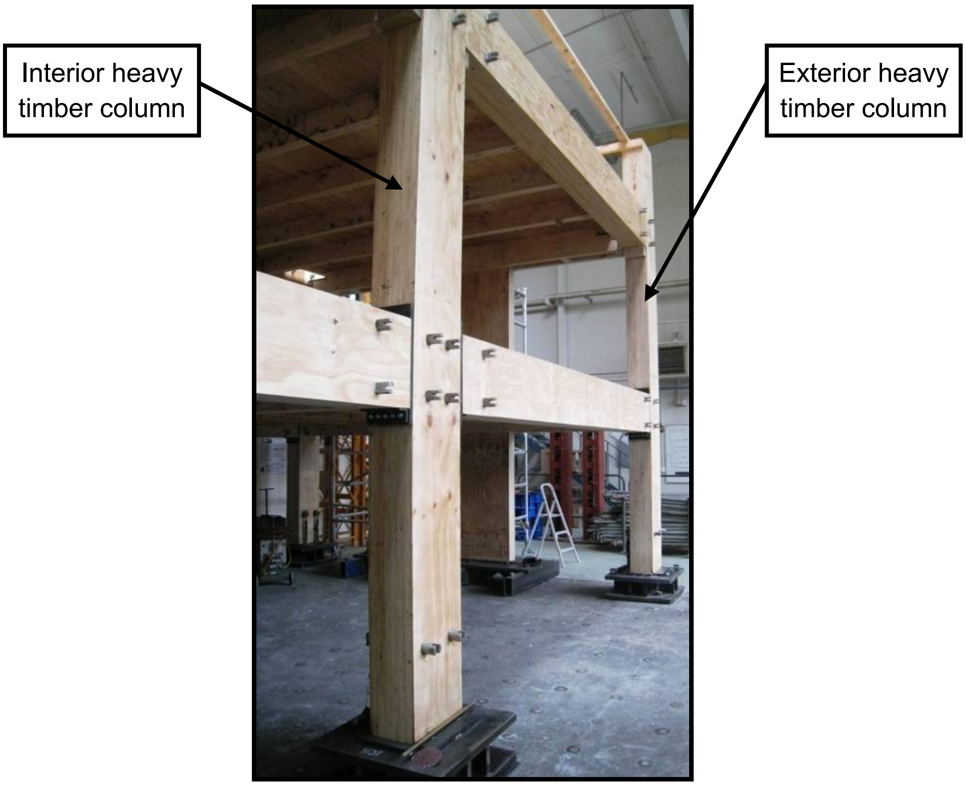

Figure 2-10 – Typical Heavy Timber Columns

Typical heavy timber columns are not equipped with energy dissipaters, however they contain epoxy grouted threaded rods at the foundation. Epoxy rods welded to the base plate act as a rigid connection and hold the heavy timber column firmly in place. For further information on heavy timber columns refer to Iqbal (2009).

2.4.3. Typical Floors

One of the distinguishing features of the pre-stressed heavy timber building is the prefabricated timber-concrete composite floor system. Heavy timber panels are produced off-site with timber joists and a nailed plywood sheet to provide formwork for the concrete slab to be poured in-situ. Coach screws embedded within grooves cut in the timber joists provide composite action for the timber-concrete connection. Further information on the timber-concrete composite floors can be found in Yeoh (2008) and O'Neill (2009). A typical floor system is shown in Figure 2-11:

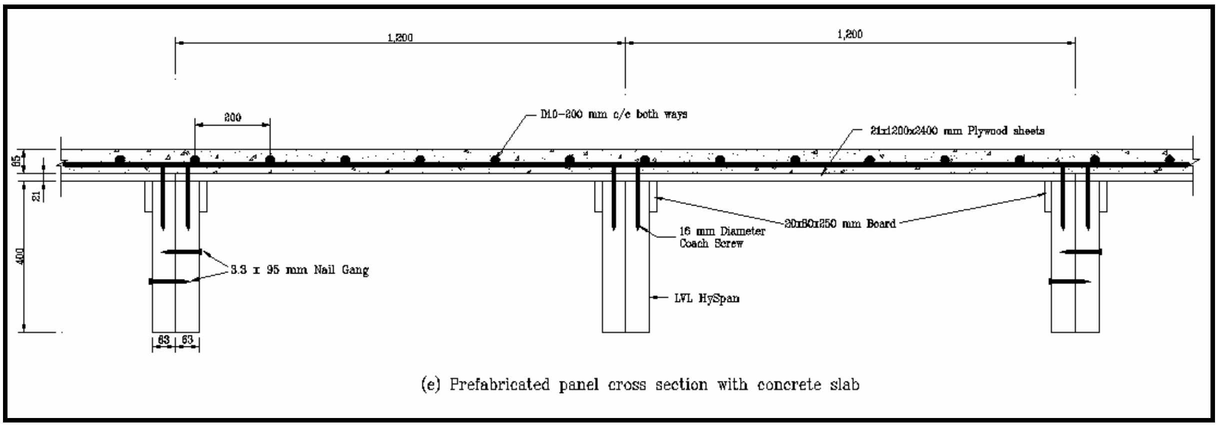

Figure 2-11 – Typical Timber-Concrete Composite Floor (Buchanan et al, 2008)

2.4.4. Typical Walls

Walls in pre-stressed heavy timber buildings provide resistance to both gravity and lateral loading. Heavy timber wall sections are pre-stressed over the height of the building to help resist overturning forces resulting from lateral loading. Lateral forces are transferred to the base through shear connections at the floor slab and the foundation. Energy dissipaters at the foundation provide an additional energy absorbing mechanism during a seismic event. A pre-stressed heavy timber wall is pictured in Figure 2-12:

Figure 2-12 – Typical Walls

2.4.5. Typical Connections

Building connections are vital for maintaining structural integrity. This is no different for pre-stressed heavy timber buildings, which present a formidable set of connection challenges due to the innovative hybrid and composite materials used by structural elements.

2.4.5.1. Joist to Beam Connection

The typical joist to beam connection provides gravity support for load transfer from the timber joists to the heavy timber beams. Prefabricated steel hangers are attached to heavy timber beams by Type 17 screws. Once the hangers are secure, the prefabricated floor system can be lowered into place. Additional information on the joist to beam connection can be found in Smith (2008b), with a typical joist to beam connection pictured in Figure 2-13:

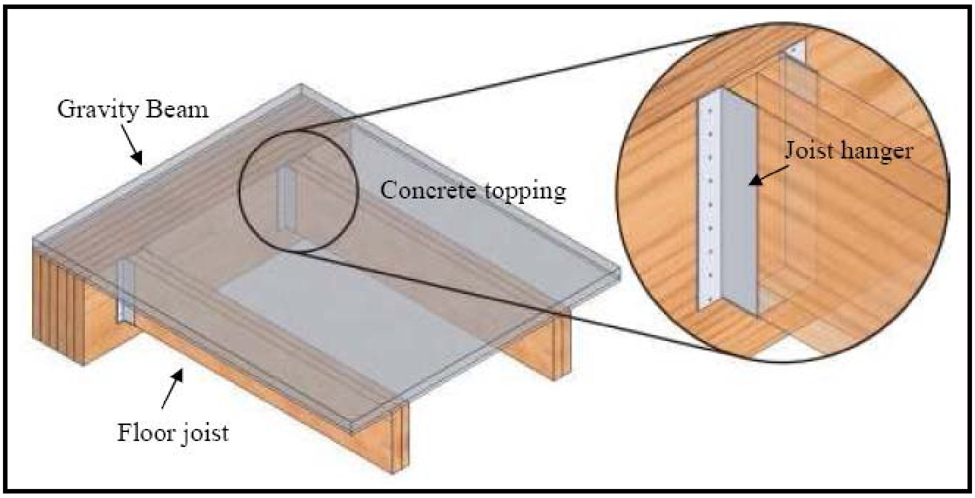

Figure 2-13 – Typical Joist to Beam Connection (Smith, 2008b)

2.4.5.2. Beam to Column Connection

The considerable size of heavy timber beam and column elements makes the typical beam to column connection difficult to design. A combination of steel threaded rods within the beam attach to the column (as seen at top in Figure 2-14) holding the beam in place until the pre-stressing can be engaged. A large steel plate between the column and beam increases the bearing area, distributing the bearing pressure along the face of the column to prevent isolated damage. Additional information can be found in Iqbal (2008, 2009) and Smith (2008b) with a typical beam to column connection shown in Figure 2-14:

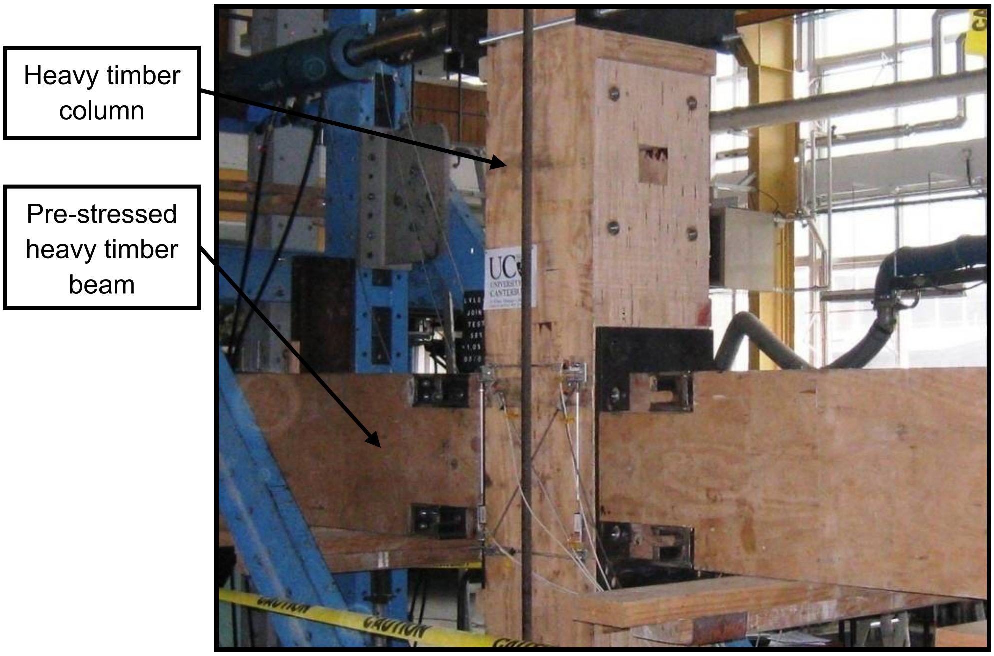

Figure 2-14 – Typical Beam to Column Connection

2.4.5.3. Slab to Frame Connection

As a prefabricated element, many connections within the composite flooring system can be assembled prior to installation. Coach screws located in notches in the timber joists provide composite action with the concrete slab. Coach bolts in the gravity beams provide composite action with the concrete slab, transferring lateral forces through shear transfer from the rigid diaphragm to the heavy timber beams. Additional information on the slab to frame connection can be found in Yeoh (2008) and O'Neill (2009) with a typical arrangement shown in Figure 2-15:

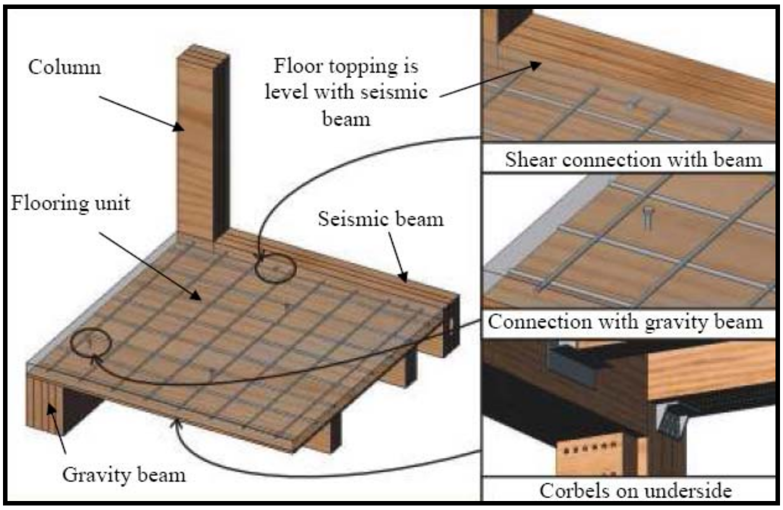

Figure 2-15 – Typical Timber-Concrete Composite Slab Connection (Smith, 2008b)

2.4.5.4. Column to Base Connection

To transfer structural loads to the ground, the heavy timber columns are connected to steel base plates firmly embedded into the foundation. Hold down bolts embedded in the concrete foundation secure the base plate for steel bars epoxy grouted into the heavy timber columns. Information on the column to base connection can be found in Iqbal (2009) and Smith (2008b). An example of the steel base plate, hold down bolts, and epoxy grouted steel bars is shown in Figure 2-16:

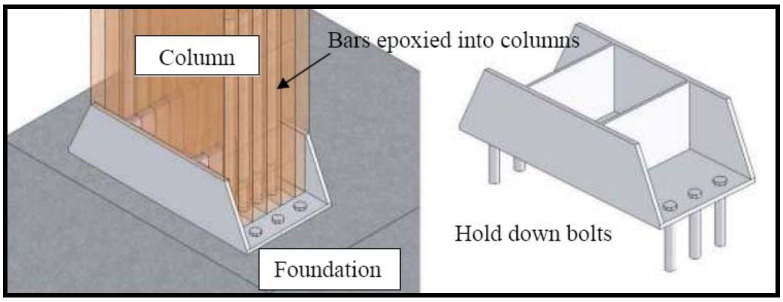

Figure 2-16 – Typical Heavy Timber Column to Base Connection (Smith, 2008b)

2.4.5.5. Wall to Base Connection

The typical wall to base connection for pre-stressed heavy timber buildings is designed similar to the column to base connection. Steel bars are epoxy grouted in the heavy timber wall and connected to the foundation using steel base plates. Both the column and wall to base connections represent epoxy adhesive connections with exposed steel at the foundation. Additional information on the wall to base connection can be found in Smith (2008b) with a typical wall shown in Figure 2-17:

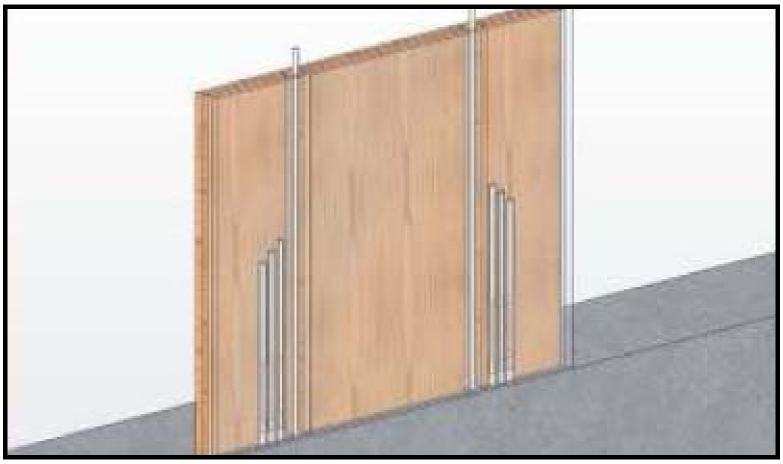

Figure 2-17 – Typical Pre-Stressed Heavy Timber Wall to Base Connection (Smith, 2008b)

2.4.5.6. Pre-Stressing Connection

Connections for pre-stressing steel are relatively simple compared to the many complex connections throughout the structure. High strength steel tendons within the pre-stressed structural element are post-tensioned and held securely in place by steel plate anchors at both ends. Pre-stressing tendons are embedded within the heavy timber elements while the steel plate anchors are exposed at the connection ends. Further information on pre-stressing connections can be found in Smith (2008b). A typical pre-stressed beam to column connection is shown in Figure 2-18:

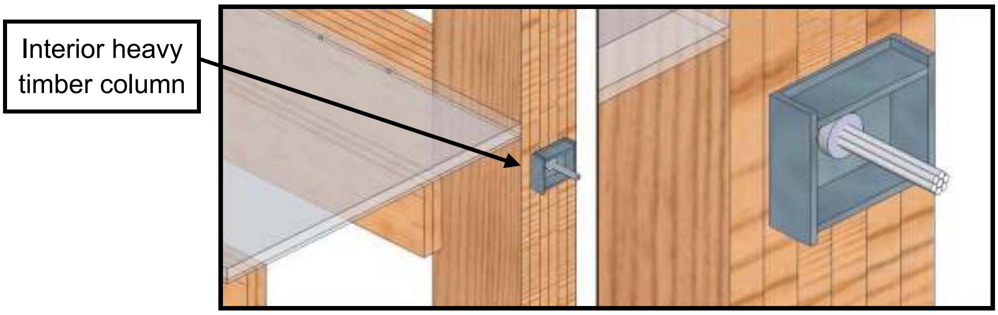

Figure 2-18 – Typical Pre-Stressing Connection (Smith, 2008b)

2.4.5.7. Energy Dissipater Connection

Pre-stressed heavy timber buildings rely on the use of a hybrid system with internal, sacrificial mild steel energy dissipaters within a heavy timber frame. Embedded energy dissipaters utilise epoxy adhesive grout with steel threaded rods to secure the dissipater into the heavy timber section. Additional information on the typical energy dissipater connection can be found in Smith (2008b). Dissipaters between the beam and column is shown in Figure 2-19:

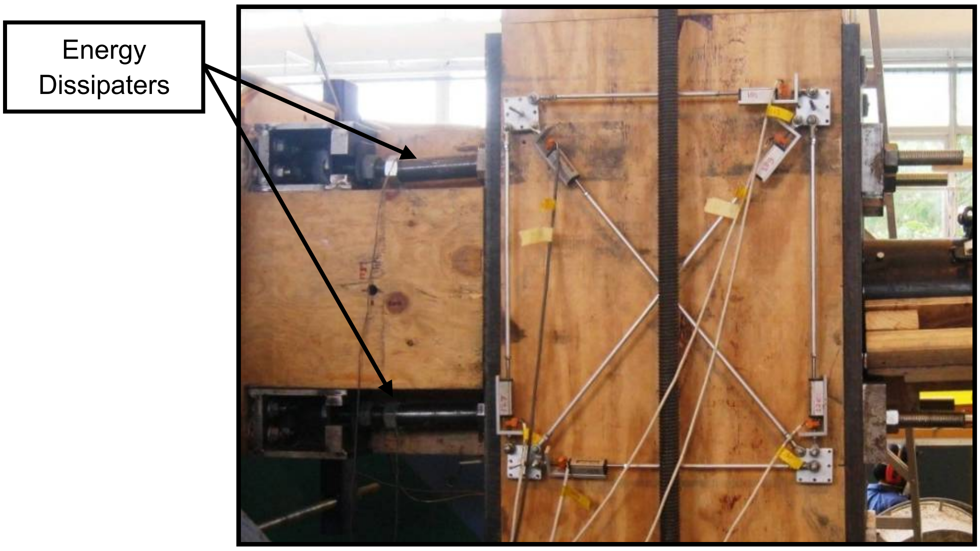

Figure 2-19 – Energy Dissipater Connection at Beam-Column Joint

2.5. Design Issues / Complications

Primary design issues include a life cycle assessment for energy and sustainability over time compared to alternative construction materials, as well as a long term performance analysis to determine the time effects on pre-stressed heavy timber members. In addition, when constructing with any timber product, a comprehensive fire resistance analysis is necessary to demonstrate the fire safety of the wood building.

2.5.1. Life Cycle Assessment

When planning the construction of any new structure, it is important to consider the life-time effects of the construction assembly. Previous research by Buchanan (2006, 2008) has highlighted the importance of a life cycle assessment in evaluating the life span of the building. Comprehensive analysis performed by Perez (2008) has engaged in comparative life cycle assessments for a case study building constructed not only in heavy timber, but also in reinforced concrete and structural steel.

Of the many life cycle assessment issues, life-time energy use in terms of building heating and cooling, embodied energy, stored carbon and CO_2 emissions are among the primary factors that must be considered. Sustainability tools can be used to compare the life-cycles for various construction materials to demonstrate potential advantages or benefits attained by a particular construction type (Perez, 2008).

2.5.2. Long Term Structural Performance

As an emerging technology, the long term effects of the structural assembly and materials must be analyzed for pre-stressed heavy timber buildings. Concerns for the durability of heavy timber buildings as well as a potential long term creep issue for pre-stressed LVL beams are two of the primary topics for consideration.

Research by Buchanan et al (2008) has evaluated the long term effects of pre-stressed heavy timber construction. Durability concerns with existing multi-storey timber framed structures have prompted the recommendation of stringent weather proofing and design and inspection procedures. Preliminary analysis with creep tests of pre-stressed LVL beams has witnessed nearly a 10% reduction of pre-stressing force due to time-dependent phenomena in the course of a year. Further analysis for both of these issues should seek to clarify the long term structural behaviour of pre-stressed heavy timber buildings.

2.5.3. Fire Behaviour / Perception Issues

A common perception exists that timber buildings are more susceptible to building fires, and thus present a greater fire hazard to building occupants compared to alternative construction materials. It is commonly believed that all timber material within a structure will contribute to the fuel load, allowing for larger and more devastating fires. Compared to reinforced concrete and structural steel, which are non-combustible, only the timber material presents the possibility of providing an ample source of fuel for a building fire. In reality, however, the charring behaviour in heavy timber sections significant improves the fire resistance and fire safety of heavy-timber structures.

This popular misconception regarding heavy timber structures suggests the need for a fire resistance analysis for the structural composition of pre-stressed heavy timber buildings. A fire resistance evaluation in this report intends to clarify the fire performance and behaviour of pre-stressed heavy timber buildings to clarify fire safety issues associated with this new construction type.

3. Fire Resistance of Pre-Stressed Heavy Timber Structural Components

Evaluating the fire resistance of pre-stressed heavy timber structural components requires a thorough understanding of the fire behaviour of the structural materials used. Building materials include the heavy timber frame, pre-stressing steel, concrete and all-purpose epoxy.

3.1. Fire Performance of Materials

It is important to have an understanding of the fire performance for the main structural materials used in pre-stressed heavy timber buildings. Structural systems utilise heavy timber for the beams, columns, floors and walls, steel for the pre-stressing plates, tendons and anchors, concrete for the timber-concrete composite floor and structural epoxy for connecting the energy dissipaters at beams, columns and walls.

3.1.1. Heavy Timber

A common misconception exists that timber framed buildings are more susceptible to fires than typical construction materials. For heavy timber making up the beam, column and frame assembly, having a larger section size actually provides advantageous fire behaviour. In fact, heavy timber structures display superior fire resistance compared to exposed structural steel and other construction materials due to the larger section sizes. The insulating charring layer formed during a fire provides considerable protection for the solid wood below (Douglas, 2005). The charring layer insulates and protects the solid wood below, preserving structural strength and integrity during a building fire.

Studies on sawn lumber and engineered wood have established an ignition temperature for wood of about 300°C (Schnabl 2005). The strength of wood steadily decreases until turning into a charring layer with zero strength at 300°C (Gerhards, 1982). The effect of temperature on the tensile strength of wood perpendicular to grain is shown in Figure 3-1. For reference, typical moisture content for timber exposed to ambient conditions is approximately 12% (Buchanan, 2001).

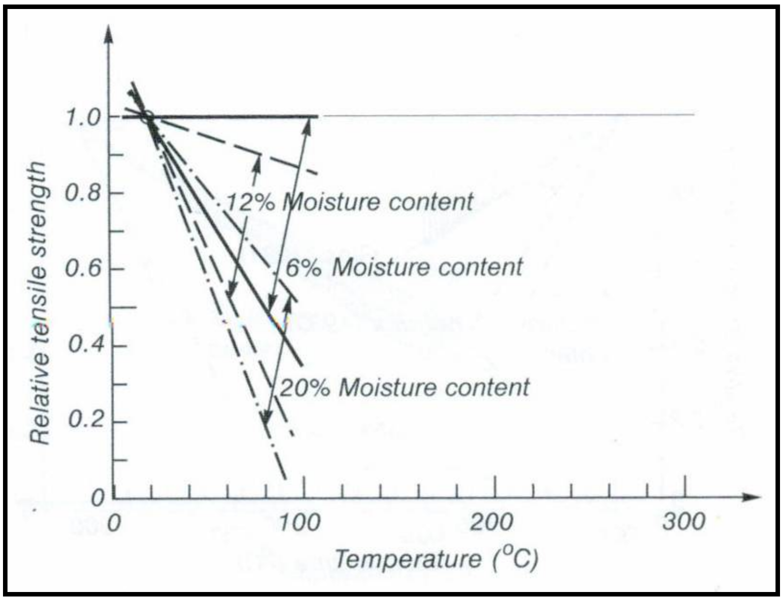

Figure 3-1 – Wood Temperature - Relative Tensile Strength (Modified from Buchanan, 2001)

Despite the strength loss with increasing temperature, heavy timber displays favourable fire behaviour. Ignited wood turns into a charring layer that insulates and protects the solid wood below. As heavy timber is heated in a building fire, the interior section of wood continues to heat, evaporating moisture at 100°C and advancing the char layer around 300°C (Buchanan, 2001). An example of the charring behaviour is shown in Figure 3-2:

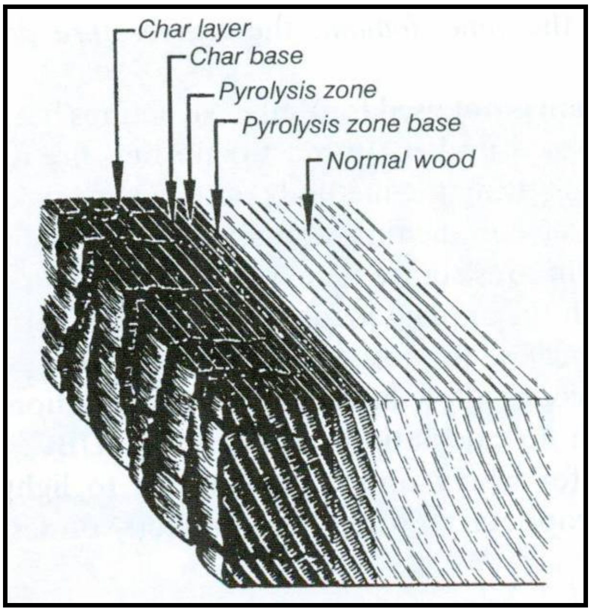

Figure 3-2 – Wood Charring Behaviour (Modified from Buchanan, 2001)

Results from charring tests provide an expected charring rate for heavy timber members (Buchanan, 2001, Douglas, 2005 and Lane, 2004). For experimental purposes, the most conservative charring rate has been selected for use in this study. Charring rates are presented in Table 3-1:

Resource	Charring Rate
Buchanan, 2001	0.50 – 0.72 mm/min
Douglas, 2005	0.63 mm/min
Lane, 2004	0.72 mm/min

Table 3-1 – Heavy Timber Charring Rates

As the charring layer advances, the cross sectional area of solid wood decreases. This results in stress increases on the remaining solid heavy timber. When the fire demand on the structural member exceeds the load capacity (resulting from reduced cross sectional area) structural failure occurs. For reference, conservative estimates assume that the charred layer and an additional 7mm of solid wood below the charring layer have zero residual strength (Buchanan, 2001).

3.1.2. Steel

When structural steel is heated to high temperatures, it experiences a significant loss in both strength and stiffness. Figure 3-3 presents a plot displaying the yield strength of structural steel, reinforcing steel and pre-stressing steel over a range of temperatures (Buchanan, 2001):

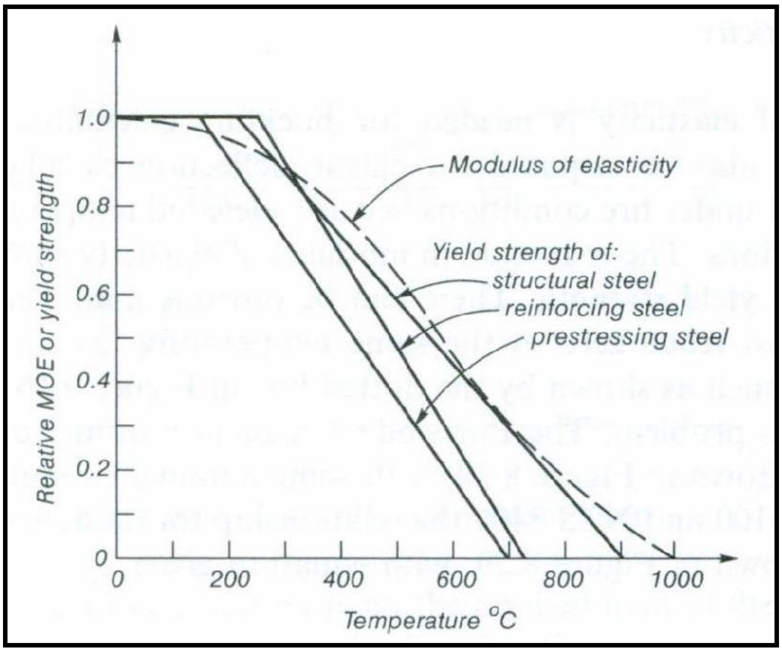

Figure 3-3 – Steel Yield Strength with Temperature (Modified from Buchanan, 2001)

As the steel temperature increases, steel elements experience a significant decrease in strength due to material softening. Equations published by Buchanan (2001) provide guidance for a limiting temperature at which both structural steel and pre-stressing steel should be limited to prevent a strength failure in structural steel elements. Each equation is specific to a given load ratio, which is the ratio of the expected load at the time of a fire divided by the load capacity of the element at cold conditions. These limiting temperature equations are presented in Equation 3-1 for structural steel and Equation 3-2 for pre-stressing steel:

$$T_{\lim} = 905 - 690 r_{load}$$ $T_{lim} = 905 - 690 r_{load}$ Structural Steel **Equation 3-1**

$$T_{\lim} = 700 - 550 r_{load}$$ $T_{lim} = 700 - 550 r_{load}$ Pre-Stressing Steel **Equation 3-2**

$T_{\lim}$ T_{lim} = The limiting temperature at which a steel member is expected to fail (°C)

r_{load} r_{load} = Load ratio (expected fire demand / cold capacity)

Providing fire protection for unprotected stele members is an important method of preventing strength loss in structural steel framing. Buchanan (2001) details many fire protection techniques including intumescent paint, spray on protection, and concrete or heavy timber encasement.

3.1.2.1. Structural Steel

Structural steel is used to make the bearing plates required for pre-stressing anchorages. These sections are expected to behave according to the temperature trends as presented above. Any exposure to fire and increase in temperature will cause softening of the member and a considerable strength loss in the exposed steel member. When the temperature of the member exceeds the critical temperature, fire protection may be necessary to maintain structural stability.

3.1.2.2. Pre-Stressing Steel

Pre-stressing steel is used in the pre-stressing tendons associated with pre-stressed heavy timber design. Pre-stressing tendons are manufactured from cold-drawn high strength steel, with yield stress exceeding 1300MPa (compared to mild steel with yield stress around 300MPa). As with all exposed steel products, an increase in temperature will result in a significant decrease in strength. This strength loss for pre-stressing steel appears more severe compared to structural steel, as seen in Figure 3-3, likely due to the chemical composition of high strength steel.

In minor fires where structural steel is subjected to high temperatures and allowed to cool to normal conditions, permanent strength loss can be expected (Gustaferro, 1988). Pre-stressing steel heated to a temperature of 480°C and allowed to cool can be expected to retain only 70% of cold strength. Above 590°C, approximately half the pre-stressing strength can be expected to be maintained.

3.1.3. Concrete

When exposed to high temperatures, concrete demonstrates considerable inherent fire resistance. As a non-combustible material with a low thermal conductivity, concrete does not burn and minimizes heat transfer throughout a concrete section; however, concrete demonstrates a strength decrease with increase in temperature. Used for its significant compressive strength despite practically zero tensile strength, a plot displaying concrete strength loss with increasing temperature is shown in Figure 3-4:

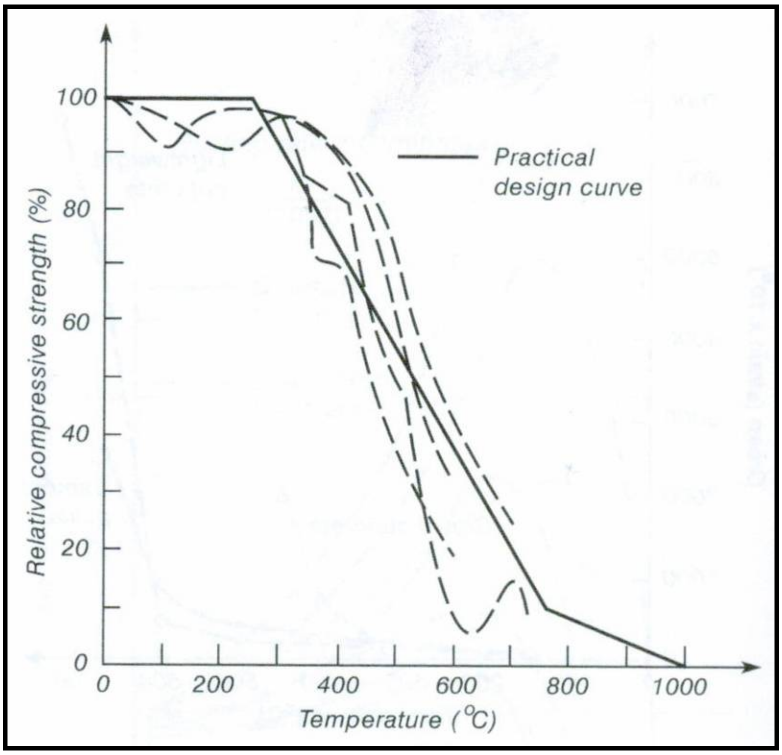

Figure 3-4 – Concrete Compressive Strength with Temperature (Modified from Buchanan, 2001)

A low thermal conductivity makes concrete a suitable insulator for protecting structural steel. A primary concern, however, occurs as the temperature of the concrete cover increases. As the temperature increases and water within the concrete evaporates at 100°C, trapped water vapour creates a pore pressure that generates tensile forces within the concrete. Having little tensile strength, this pore pressure causes spalling of the concrete cover, with large chunks of concrete being violently separated from the structural member (Buchanan, 2001). When spalling occurs, the cover may be compromised. This could expose reinforced steel to high temperatures and result in significant strength loss for the structural element.

3.1.4. Structural Epoxy

The need for a steel to wood connection solution has increased the popularity of structural epoxy as a viable connection tool. Typically all-purpose epoxies are used for their ability to provide tensile strength with different material types. The introduction of high temperature epoxies has broadened the range of epoxy use, allowing for the use of high temperature epoxy for connections that may be exposed to high temperatures.

3.1.3.1. All-Purpose Epoxy

All-purpose epoxies display considerable strength at ambient conditions. Heating effects, however, are often overlooked. Commonly used as an epoxy adhesive grout for steel rod into concrete, the favourable insulation behaviour with concrete embedment prevents significant heat transfer into the epoxy connection. Embedment in concrete protects the epoxy from exposure to high temperatures associated with building fire scenarios.

When considering the exposure of all-purpose epoxy to high temperatures, however, strength losses are apparent. Embedment in heavy timber instead of concrete means insulation performance will not be as effective for preventing heat transfer. Research by Barber (1994) on the temperature effects on all-purpose epoxy demonstrated significant strength loss with increasing temperature, as shown in Figure 3-5:

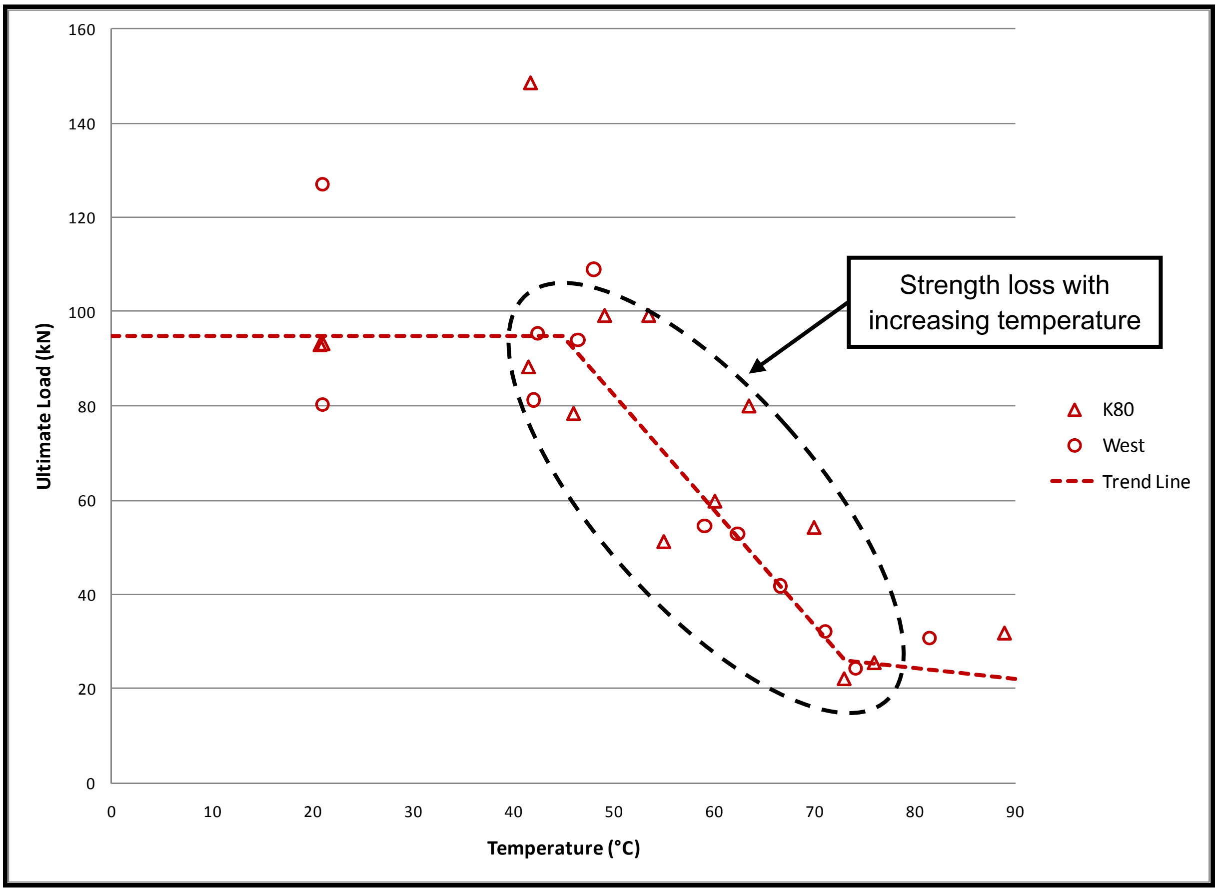

Figure 3-5 – All-Purpose Epoxy Ultimate Load with Temperature (Barber, 1994)

Results from Barber (1994) suggest that all-purpose epoxy reaches a strength plateau and has no significant strength beyond 100°C.

3.1.3.2. High Temperature Epoxy

One solution to the all-purpose epoxy adhesive strength loss at high temperatures is the use of high temperature epoxy. Previous high temperature epoxies required curing at high temperatures (Harris, 2004). Current high temperature epoxies, however, cure at ambient temperatures. This makes their use in construction more viable and practical. Testing with high temperature epoxy is discussed in greater detail in Chapter 5 through Chapter 8.

3.2. Fire Resistance of Structural Members

Calculating the fire resistance for structural members establishes a conservative amount of time for which the structural component can be expected to meet performance requirements. Methods provided by Buchanan (2001) suggest a fire demand versus capacity analysis to determine the fire resistance for structural elements. The amount of time the member capacity exceeds fire demand is the fire resistance time in the standard fire for the structural element.

Fire resistance calculations are limited to the strength domain for two reasons. First, fire resistance is not required to be evaluated for the integrity and insulation criteria for structural beams and columns as these are purely load-bearing elements. Second, extensive full-scale testing would be necessary to quantify the integrity and insulation properties for the pre-stressed heavy timber floor and wall systems.

A representative heavy timber case study has been used for evaluating the fire resistance of pre-stressed heavy timber structures. Structural details including demand loadings, tributary areas and member sizes can be found in the framing plan shown in Figure 3-6 (Smith, 2008b):

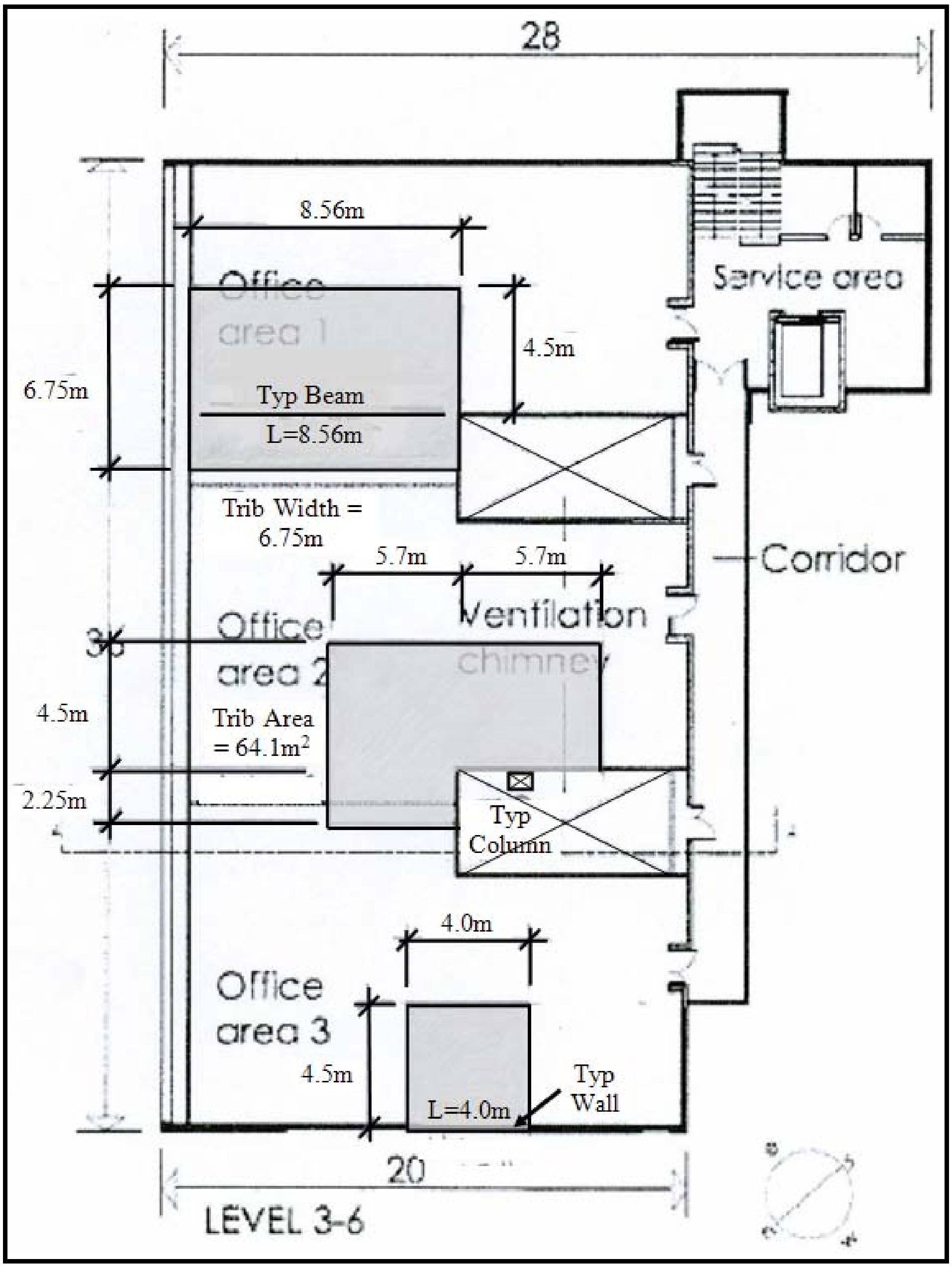

Figure 3-6 – Typical Structural Framing Plan

3.2.1. Typical Beams

Typical pre-stressed heavy timber beams are composed of hollow laminated veneer lumber (LVL) sections with embedded unbonded pre-stressing steel. Only the LVL has been considered for calculating the fire resistance, as the embedded pre-stressing is assumed to not contribute any significant fire resistance to the assembly. Analysis considered three sided exposure for the typical pre-stressed heavy timber beam, with the top protected by the composite floor system above. Typical heavy timber beams have an exterior dimension of 378mm wide by 600mm deep, with the interior hollow cross section at mid-span equal to 252mm wide by 410mm deep (Smith, 2008b) as shown in Figure 3-7:

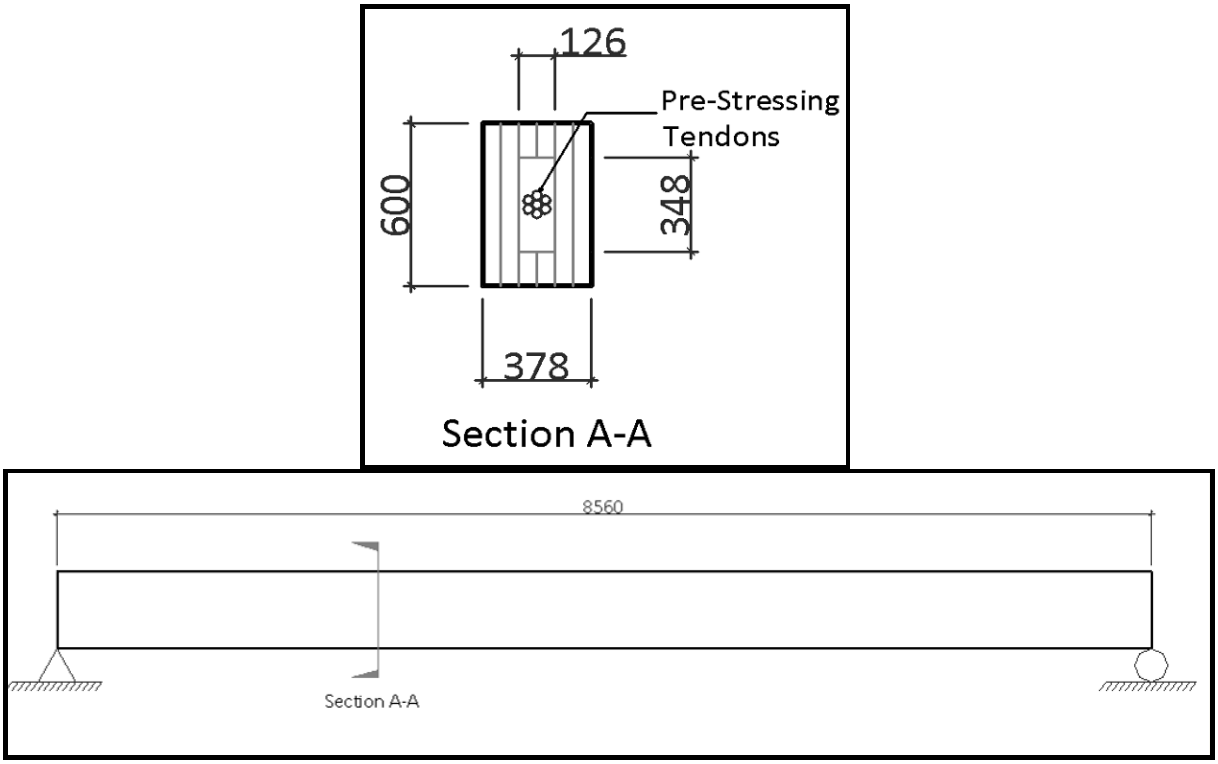

Figure 3-7 – Typical Pre-Stressed Heavy Timber Beam

Framing plans from Smith (2008b) for the case study building provide design loads, capacity values and strength properties for cold design that can be extended to fire design (see Figure 3-6 and Appendix 1). Demand versus capacity was evaluated at one minute intervals to determine the fire resistance time for pre-stressed heavy timber beams.

3.2.1.1. Fire Resistance Calculation

The fire resistance analysis for typical beams considered the cross section of LVL providing strength resisting the moment demand at beam mid-span. Demand versus capacity equations involving cold design were first investigated to obtain a cold capacity. This method was extended to a fire design to determine the fire resistance for the structural element. The demand versus capacity equation was evaluated according to Equation 3-3 provided by Buchanan (2001):

$$M^{*}_{fire} \leq M_{fire} \qquad \textbf{Equation 3-3}$$

M^{*}_{fire} = Moment demand at beam mid-span in the fire limit state (kNm)

M_{fire} = Moment capacity at beam mid-span in the fire limit state (kNm)

The demand/capacity ratio was calculated at one minute intervals to evaluate the beam demand versus capacity. The charring rate was held constant throughout the analysis. As charring continued to reduce the beam cross sectional area, and thus the section modulus, the beam would lose moment capacity. Sufficient charring would cause the demand to be greater than the capacity, meaning the heavy timber beam would fail Equation 3-3 above. The amount of time to a structural stability failure represents the fire resistance time for the connection.

Analysis in Appendix 1 provides the fire resistance calculation for heavy timber beams. A fire resistance calculation for the stability domain suggests a fire resistance time of 2 hours and 39 minutes (159 minutes) for typical pre-stressed heavy timber beams.

3.2.1.2. Fire Resistance Analysis

The fire resistance calculation suggests that typical pre-stressed heavy timber beams are able to sustain the fire demand load for 159 minutes prior to structural stability failure. The LVL cross section for typical beams offers favourable fire performance, as the fire resistance value provides more than a 120 minute fire rating for a multi-storey heavy timber building.

Inherent fire resistance was calculated from the performance of the solid wood section alone. The depth of the LVL is assumed to provide the entire inherent fire resistance for the beam, as contributions from the pre-stressing steel and energy dissipaters are assumed to be negligible even though embedded pre-stressing steel tendons near the centre of the beam cross section are assumed to remain at ambient temperature until the beam has nearly completely charred.

Epoxy grouted steel rod energy dissipaters will be protected as described further in the report. Exposed, unprotected steel energy dissipaters will have negligible fire resistance, which is acceptable as a major earthquake is considered an extreme event, and is not expected during a fire event. Energy dissipaters are easily replaced following a minor fire in preparation for a possible major earthquake.

Other exposed, unprotected steel products such as steel bearing plates and pre-stressing anchors, however, require additional fire protection. These elements are required to maintain their structural integrity, as the pre-stressing is intended to hold the structure together at all times. Exposed steel components should be provided with fire protection so they have sufficient fire resistance time. A survey of structural fire protection techniques can be found in Buchanan (2001).

3.2.2. Typical Columns

Typical heavy timber columns are composed of solid sections of LVL with embedded mild steel energy dissipaters located at the foundation as part of the hybrid system. The columns themselves are not pre-stressed because the horizontal pre-stressing in the beams and the vertical pre-stressing in the walls is sufficient to maintain structural integrity. If the columns are pre-stressed, the primary concern would be the fire resistance of the anchorage, as with the beams.

Only the LVL has been considered for the fire resistance calculation, as the energy dissipaters are only active during extreme lateral loading such as earthquakes. Typical heavy timber columns have a cross section of 378mm wide by 600mm deep, as shown in Figure 3-8:

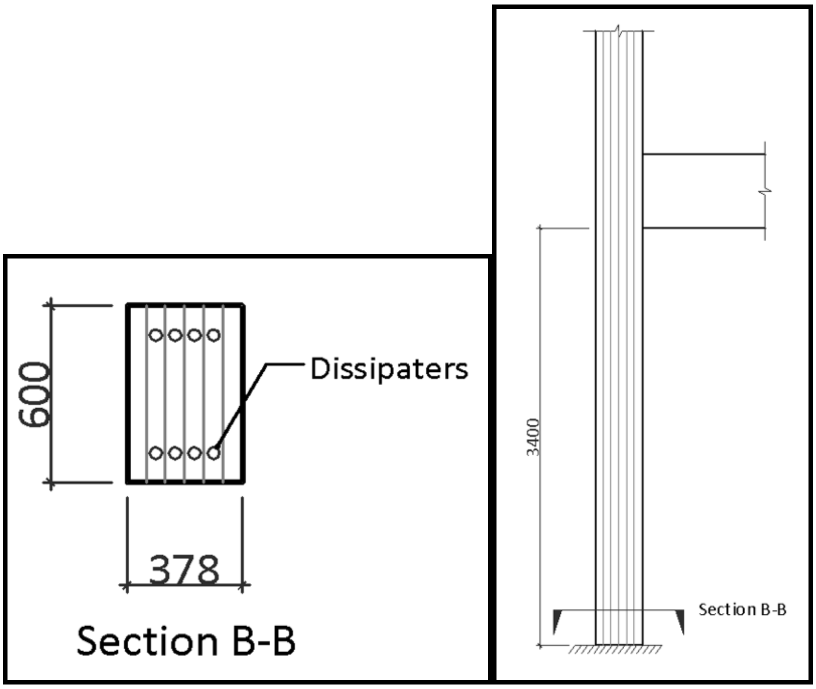

Figure 3-8 – Typical Heavy Timber Column

Evaluating the fire resistance for typical heavy timber columns requires loading and strength properties. A typical floor plan showing tributary loading for columns is shown in Figure 3-6. Typical column strength and loading properties are provided in Appendix 2.

The fire resistance for heavy timber columns was evaluated purely in the strength domain. As charring occurs the cross sectional area decreases, reducing the axial strength of the structural member. The fire resistance is the amount of time that the axial capacity is greater than the axial demand in the fire limit state. The axial capacity under fire conditions was calculated in one minute intervals to determine the fire resistance for typical pre-stressed heavy timber columns.

3.2.2.1. Fire Resistance Calculation

The fire resistance calculation for heavy timber columns was evaluated in two methods. The first involved determining the fire resistance for typical columns checking for axial demand versus capacity. The second involved evaluating the fire resistance for heavy timber columns for column buckling. The lesser of the two fire resistance values will be the governing fire resistance for typical heavy timber columns.

Checking for axial strength, the fire resistance calculation for pre-stressed heavy timber columns evaluated the cross section of LVL. An axial demand versus capacity evaluation was first made at cold conditions to confirm the structural design and extended to fire conditions for the fire resistance calculation. The demand versus capacity analysis was performed by checking the axial fire demand versus capacity at the mid-section of the first floor column supporting the greatest amount of load. The demand versus capacity equation is shown in Equation 3-4 (Buchanan, 2001):

$$N^*_{fire} \leq N_{fire} \qquad \textbf{Equation 3-4}$$

N^*_{fire} = Axial demand at column mid-section in the fire limit state (kN)

N_{fire} = Axial capacity at column mid-section in the fire limit state (kN)

Charring was assumed to occur at a constant rate to reduce the cross section for heavy timber columns. As the column experiences charring, it loses axial strength due to a reduction in cross sectional area. Once the charring depth reaches a critical limit, the remaining timber is unable to sustain the axial demand and the typical column fails Equation 3-4 above. The amount of time to structural failure is equivalent to the fire resistance time for the typical column.

The second criteria for determining the fire resistance for typical heavy timber columns involved checking a typical column for column buckling. Structural design requires that columns maintain an effective height to width ratio within conservative limits. Restrictions are imposed to prevent buckling under axial load. The column buckling equation is shown in Equation 3-5:

$$\frac{kl}{r} \leq 200 \; \frac{kl}{r} \leq 200 \qquad \textbf{Equation 3-5}$$

k k = Effective length factor (Assume 1.0 to be conservative)

l = Unbraced column length (m)

r r = Radius of gyration (m)

The slenderness ratio was evaluated at one minute intervals. Charring was assumed to occur at a constant rate. As the charring layer increases, the cross section, and thus the radius of gyration, of the typical heavy timber column decreases. The fire resistance time is the amount of time that the typical heavy timber column satisfies the column buckling equation presented above.

The fire resistance calculation for typical heavy timber columns is shown in Appendix 2 for axial strength and column buckling. This analysis displays the charring depth over time and the strength effects on the section checking both axial strength and column buckling. The fire resistance calculation for axial strength reveals nearly a 3 hour (178 minute) fire resistance time for typical pre-stressed heavy timber columns. The fire resistance calculation for column buckling reveals a 211 minute fire resistance time.

The governing fire resistance for typical columns is the governing value. A fire resistance analysis checking for axial strength suggests a governing fire resistance of nearly 3 hours for typical heavy timber columns.

3.2.2.2. Fire Resistance Analysis

Typical columns can be expected to resist an axial fire demand load for nearly 3 hours with exposure to the standard fire. This calculation has considered the inherent fire resistance of LVL only, checking for axial strength and column buckling and neglecting the embedded energy dissipaters and epoxy grouted steel elements.

The fire resistance calculation suggests the LVL cross section for typical columns provides considerable fire resistance. Further evaluation of the embedded epoxy grouted steel threaded rods, however, is necessary for thorough analysis. This evaluation is shown in Section 3.3.4 for epoxy grouted connections.

3.2.3. Typical Floors

Typical floors provide the primary system for gravity load transfer to structural beams and columns. As a timber-concrete composite system, they provide unique performance and behaviour compared to traditional flooring solutions. Typical floors for pre-stressed heavy timber buildings have been subjected to experimentation and testing by Buchanan et al (2008) and Yeoh (2008). Research by O'Neill (2009) provides a comprehensive quantitative and qualitative evaluation for timber-concrete composite floors in pre-stressed heavy timber structures.

3.2.4. Typical Walls

Typical pre-stressed heavy timber walls consist of solid LVL sections with embedded pre-stressing steel and energy dissipaters. The LVL cross-section has been considered for fire resistance analysis, as pre-stressing steel makes no significant contribution to fire resistance. Furthermore, the energy dissipaters are intended for seismic loadings as opposed to fire loadings. A typical exterior wall cross section is 4000mm wide by 252mm deep, as shown in Figure 3-9:

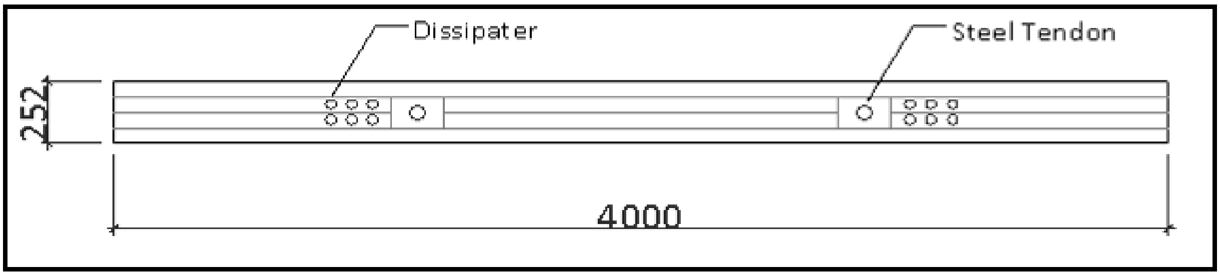

Figure 3-9 – Typical Pre-Stressed Heavy Timber Wall

Typical walls transfer gravity loads to the foundation, with the first floor wall carrying the greatest axial demand. The worst-case gravity loading has been considered for a typical wall located at the first floor. Structural framing plans for the typical wall are shown in Figure 3-6. Strength and material values are provided in Appendix 3. The fire resistance calculation was limited to the strength domain, evaluating gravity forces only for the typical first floor pre-stressed heavy timber wall.

The fire resistance is the amount of time for which the typical wall is able to sustain an axial fire demand load. The demand/capacity ratio has been calculated at one minute intervals to evaluate the structural stability of the typical wall section. The amount of time the typical wall axial capacity is greater than the axial fire demand is the fire resistance for the pre-stressed heavy timber wall.

3.2.4.1. Fire Resistance Calculation

The fire resistance calculation for typical pre-stressed heavy timber walls is based on the cross sectional area of LVL for typical walls. The cross sectional area provides axial strength to resist gravity loads from above. An axial demand versus capacity analysis beginning with cold design through to a fire calculation provides the fire resistance time for the typical wall. The fire resistance calculation is found in Equation 3-6 (Buchanan, 2001):

$$N^*_{fire} \leq N_{fire} \qquad \textbf{Equation 3-6}$$

N^*_{fire} = Axial demand at wall mid-section in the fire limit state (kN)

N_{fire} = Axial capacity at wall mid-section in the fire limit state (kN)

The cross sectional area providing the axial strength was assumed to experience charring at a constant rate. Once a critical cross section was reached, the typical wall failed Equation 3-4, presented above. The amount of time the typical structural wall was able to resist the axial load in the fire limit state is equal to the fire resistance time for the structural component.

A fire resistance calculation for pre-stressed heavy timber walls is found in Appendix 3. The analysis displays the material properties as well as the axial capacity of the section throughout the duration of fire exposure. The fire resistance calculation shows a 2 hour and 43 minute (163 minute) fire resistance time for typical heavy timber walls.

3.2.4.2. Fire Resistance Analysis

Structural walls can be expected to resist an axial demand with exposure to the standard fire for more than two and half hours before a structural stability failure occurs. Despite the favourable fire resistance associated with heavy timber, steel elements used with typical walls must also be considered. Embedded pre-stressing steel elements and epoxy grouted steel rod energy dissipaters would need to be sufficiently protected by the surrounding heavy timber or provided with applied fire protection.

3.3 Fire Resistance of Structural Connections

A fire resistance analysis for typical connections is intended to establish the fire performance of screwed connections, bolted connections, pre-stressing connections and epoxy grouted steel rod connections in pre-stressed heavy timber buildings.

3.3.1. Screwed Connections

Typical screwed connections in timber construction rely on the redundancy of screws for strength and ductility. Screwed connections in pre-stressed heavy timber are no different. A significant number of screws located at the joist hanger, corbels, and slab connections provide connection strength for structural elements. In general, screwed connections provide excellent structural performance. At high temperatures, however, screwed connections display poor fire performance due to the large surface area of screw heads exposed to fires.

3.3.1.1. Fire Resistance Calculation

Buchanan (2001) suggests that screwed connection behaviour in timber construction is identical to nailed connection behaviour. Results from nailed tests have been assumed for the screwed connection analysis. Testing performed by Noren (1996) demonstrated the fire performance of nailed connections subjected to the ISO 834 standard fire curve. Figure 3-10 displays testing results presented in Buchanan (2001) for nailed connection time to failure subjected to the standard fire for a range of load ratios:

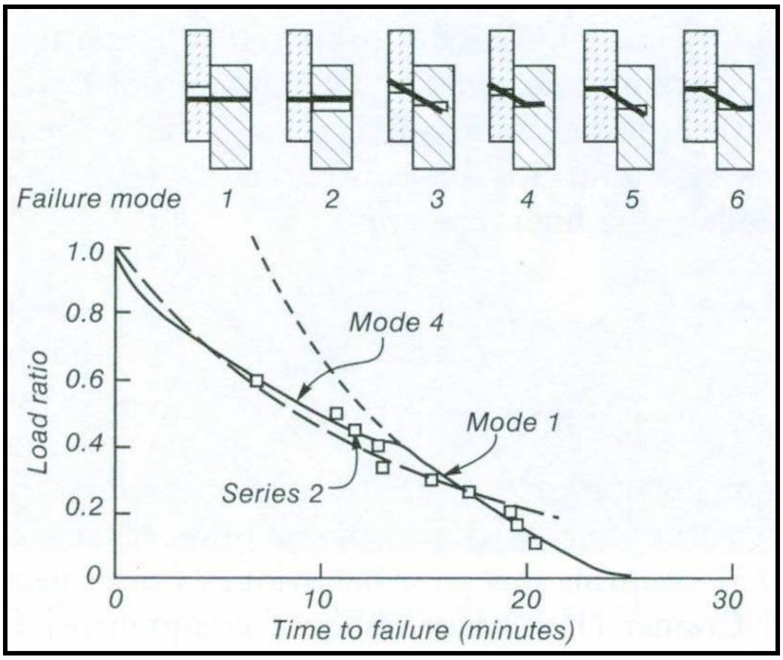

Figure 3-10 – Screw Fire Resistance (Modified from Buchanan, 2001)

The connection time to failure in the ISO 834 standard fire is equal to the fire resistance time for the connection. Based on testing data provided by Noren (1996), the fire resistance times range from 0 to 20 minutes for typical screwed connections.

3.3.1.2. Fire Resistance Analysis

In a building fire, localized failure of screwed connections may occur adjacent to the fire plume. This would cause localized failure of screws. It can be expected, however, that considerable redundancy of screws throughout a structural element will compensate for isolated screws that have failed under fire conditions. This behaviour suggests that the existing exposed screwed connection associated with current pre-stressed heavy timber construction is acceptable for design.

3.3.2. Bolted Connections

Typical bolted connections for pre-stressed heavy timber buildings occur at the pre-stressed beams to heavy timber column connections. Bolts located at the top and bottom of the beam thread through a steel plate to transfer vertical gravity loads to the column. Bolts also hold the beam in place in preparation for post-tensioning of pre-stressed elements.

3.3.2.1. Fire Resistance Calculation

A series of experimental studies performed by Lau (2006) evaluated the fire resistance of bolted connections in LVL. These studies produced fire resistance times for wood to steel to wood connections held at a constant tensile load and subjected to the ISO 834 standard fire curve. The testing arrangement for wood to steel to wood connections, similar to the wood beam to steel plate to wood column, is shown in Figure 3-11:

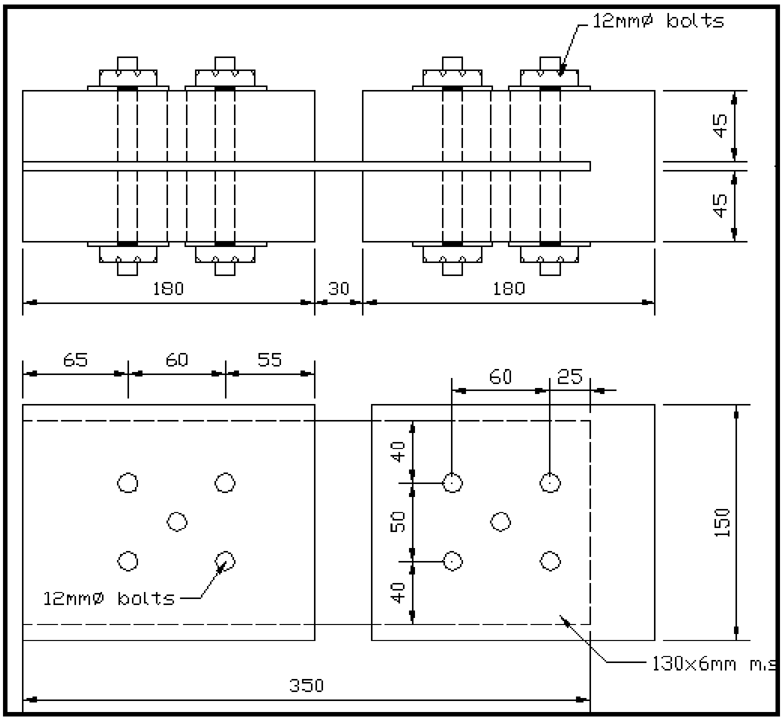

Figure 3-11 – Wood Steel Wood Experimental Set-Up (Lau, 2006)

Testing results suggest an average fire resistance time of 16.3 minutes for wood to steel to wood connections (Lau, 2006). Results suggest that the cross section of wood providing cover for the steel plate may have a significant influence in connection strength. A larger heavy timber cross section and cover depth could provide improved fire performance.

3.3.2.2. Fire Resistance Analysis

A fire resistance analysis suggests less than 17 minute fire resistance for steel to wood to steel connections. Given the experimental cross section of cover compared to that for pre-stressed beams and columns, fire resistance times would be expected to be significantly improved for pre-stressed heavy timber construction. Further, providing fire protection for any exposed steel elements would improve the connection fire resistance.

3.3.3. Pre-Stressing Connections

One of the primary features of pre-stressed heavy timber construction is the addition of pre-stressing elements. Pre-stressing tendons are embedded in the heavy timber beams and walls to maintain post-tensioning loads. An extensive literature review has not yielded any references evaluating the fire resistance of pre-stressed connections. This is likely due to the fact that pre-stressing fire resistance is not considered in reinforced-concrete construction, as pre-stressing steel is embedded in concrete and protected in fire. Use in pre-stressed heavy timber structures, however, makes pre-stressing fire resistance a credible consideration.

3.3.3.1. Fire Resistance Calculation

Structural steel displays excellent structural behaviour but extremely poor fire behaviour, as previously discussed in Section 3.1.2. Exposed steel elements display significant strength loss with exposure to high temperatures. This is especially true for pre-stressing steel elements, as they suffer from permanent strength loss when heated to high temperatures.

As embedded elements within heavy timber beams and walls, it can be assumed that pre-stressing tendons are safely protected from high temperatures. Exposed steel plates and anchors, however, present a significant problem. A conservative fire resistance time for exposed steel plates and anchors suggests these exposed steel elements have zero fire resistance (Buchanan, 2001).

3.3.3.2. Fire Resistance Analysis

Significant strength loss for exposed steel at high temperatures suggests adequate fire protection is required to prevent temperature increase and loss of strength. Providing sufficient fire protection for exposed pre-stressing anchors and plates will improve the fire resistance for the typical pre-stressing connection. Fire protection solutions can be found in Buchanan (2001).

3.3.4. Epoxy Grouted Steel Rod Connections

Epoxy grouted steel rods, used for the energy dissipaters, are a primary component of the hybrid design for pre-stressed heavy timber structures. Connecting the mild steel energy dissipaters to laminated veneer lumber sections requires the use of epoxy grouted steel threaded rod connections. A high strength steel rod is grouted to both ends of the structural element, with the energy dissipater screwed into either free end. Figure 3-12 presents a group of epoxy grouted steel threaded rods at a beam-column connection:

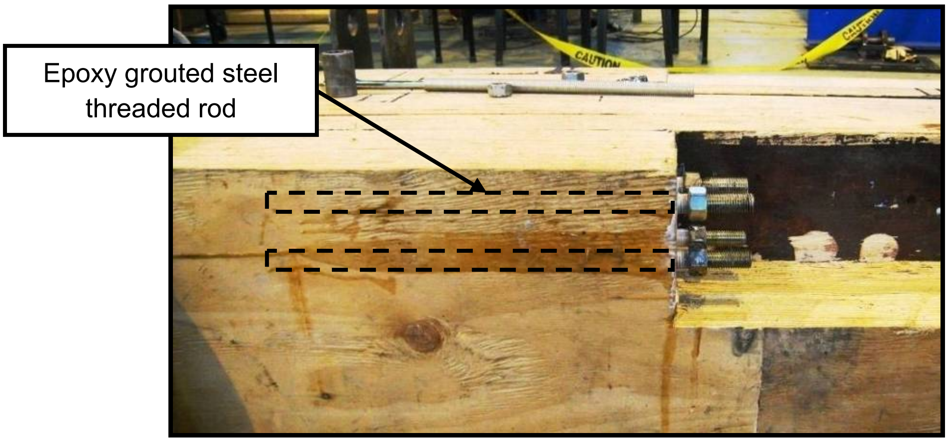

Figure 3-12 – Epoxy Grouted Steel Rod Connection in Beam

All-purpose epoxy has been used in typical epoxy grouted steel rod connections for pre-stressed heavy timber buildings. Additional experimentation with high temperature epoxy is intended demonstrate the fire performance, behaviour and resistance of high temperature epoxy for use in future pre-stressed heavy timber buildings.

3.3.4.1. Fire Resistance Calculation

Previous testing performed by Barber (1994) and Harris (2004) has provided data regarding the fire performance of epoxy grouted steel rods in engineered wood. Tensile testing of epoxy grouted steel rod specimens demonstrated that all-purpose epoxy may not maintain any considerable strength beyond 100°C (Barber, 1994). Further testing with all-purpose epoxy grouted steel rod samples in LVL subjected to the ISO 834 standard fire produced a set of fire resistance times for this unique connection (Harris, 2004). Fire resistance times are shown in Table 3-1:

Specimen Size (mm)	Mean Calculated Fire Resistance Time (min)
63 x 63	11.9
105 x 105	35.2

Table 3-2 – Epoxy Grouted Steel Rod Fire Resistance (Harris, 2004)

The fire resistance calculation displays that a typical 63mm square section was able to resist a tensile demand load for nearly 12 minutes, with the larger 105mm square section resisting the load nearly 3 times as long, for 35 minutes. Results indicate that the cross section of laminated veneer lumber, providing cover for the centrally located epoxy grouted steel rod specimen, has a significant impact on the fire resistance of the connection.

3.3.4.2. Fire Resistance Analysis

Test results suggest that fire performance appears to be significantly affected by the cross sectional area and cover depth of heavy timber protecting the embedded steel rod. Increasing the cross section from 63mm square to 105mm square significantly increased the fire resistance time. The considerably larger cross section associated with heavy timber beams and columns suggests that sufficient cover depth will be necessary for protecting the epoxy grouted steel rod connection.

The prospect of using high temperature epoxy in place of all-purpose epoxy presents a potential improvement for the fire resistance of the connection. Further testing of high temperature epoxy is necessary to thoroughly evaluate the feasibility this product may have for use in typical epoxy grouted connections in pre-stressed heavy timber buildings.

3.4. Fire Resistance Conclusions

The case study building provided member details necessary for a fire resistance evaluation of typical pre-stressed heavy timber structural components. These include the typical beams, columns, floors, walls and connections. While all structural heavy timber elements provide fire resistance times greater than 120 minutes, additional fire protection is necessary at critical locations throughout the structure. Exposed steel elements used for the pre-stressing steel connection require additional fire protection to achieve a fire resistance comparable to structural components.

3.4.1. Typical Beams

A fire resistance analysis for typical pre-stressed heavy timber beams considered the cross sectional area of the hollow laminated veneer lumber only. The pre-stressing steel elements contribute negligible fire resistance to the structural component. With a fire resistance time of 2 hours and 39 minutes, the typical beam demonstrated the governing fire resistance for structural components. While the cross sectional area of heavy timber provides considerable fire resistance for the structural beam, fire protection for exposed steel elements is critical to maintaining the structural stability for the connection.

3.4.2. Typical Columns

Typical columns display the greatest fire resistance time for all structural components. Solid laminated veneer lumber columns in pre-stressed heavy timber buildings provide a 2 hour and 58 minute fire resistance time, making the columns the most structurally stable fire design element in a pre-stressed heavy timber structure. This is important as columns are critical to providing global structural support at each level. Providing fire protection for all exposed steel elements, however, is necessary to prevent fire damage to structural components.

3.4.3. Typical Floors

A fire resistance analysis on timber-concrete composite floors can be found in research by O'Neill (2009). On a qualitative level however, the timber-concrete composite is expected to display excellent behaviour, as concrete is not combustible and has very low thermal conductivity. Additional information on timber-concrete composite floors can also be found in Yeoh (2008).

3.4.4. Typical Connections

Typical connections in pre-stressed heavy timber buildings utilise steel products for connection solutions. Steel is used for screwed, bolted and pre-stressing connections. The poor fire performance of exposed steel makes fire protection a requirement for exposed steel connections. Studies on all-purpose epoxy grout (Barber, 1994 and Harris, 2004) suggest poor fire performance for epoxy grouted steel rods in heavy timber. Alternative solutions may achieve greater fire performance.

3.3.4.1. Screwed Connections

Typical screwed connections in pre-stressed heavy timber construction provide connections for joist hangers, attaching corbels for beam bearing, and within the slab connection. A fire resistance analysis suggests screwed connections display poor fire performance with a fire resistance time between 0 and 20 minutes. The redundancy of screwed elements and the considerable number of screws associated with connection design, however, suggests typical screwed connections are valid for design.

3.3.4.2. Bolted Connections

Previous fire resistance analysis with bolted connections demonstrated a fire resistance time of nearly 17 minutes for bolted wood to steel to wood connections. Providing additional fire protection for typical bolted connections should improve the performance and structural fire resistance for the connection.

3.3.4.3. Pre-Stressing Connections

Pre-stressing connections in pre-stressed heavy timber buildings involve the combination of pre-stressing tendons and steel plates and anchors. Pre-stressing tendons are embedded in heavy timber beams and walls while steel plates and anchors are exposed at either end. Fire protection is required to provide sufficient fire resistance for all exposed steel elements.

3.3.4.4. Epoxy Grouted Steel Rod Connections

Epoxy grouted steel rod connections are used at locations in which seismic dampening with energy dissipaters is necessary. All-purpose epoxy grout is traditionally used for energy dissipater installation into heavy timber elements. A fire resistance analysis suggests all-purpose epoxies demonstrate poor performance at high temperatures.

Future analysis could evaluate the feasibility of using high temperature epoxy in place of all-purpose epoxy for the heavy timber-steel rod connection. Additional testing of high temperature epoxy will serve to determine epoxy behaviour and performance in fire conditions for use in pre-stressed heavy timber buildings.

4. Materials and Testing

This section provides a broad overview of testing materials, testing machinery and details for the preparation of test specimens. Testing materials consist of laminated veneer lumber (LVL) engineered wood and steel threaded rod for the steel to wood connection. Testing machinery for the tensile pull out tests includes the Avery Testing Machine and custom furnace, located in the University of Canterbury Civil Engineering and Chemical and Process Engineering Laboratory, respectively.

4.1. Laminated Veneer Lumber (LVL)

Laminated veneer lumber (LVL) is a popular engineered wood product selected for use in construction because of greater strength and stiffness compared to sawn lumber. LVL consists of thin wood veneers glued together to form beams and columns for timber framed construction (as seen in Figure 4-1). As an engineered product, LVL is preferred to sawn lumber because inconsistencies in wood are removed. This allows for greater uniformity and structural performance.

Figure 4-1 – NelsonPine LVL Column

LVL for the project was provided by NelsonPine Industries Ltd, produced at the NelsonPine plant on the shores of Tasman Bay. NelsonPine LVL is made from 3mm veneer sections of Radiata Pine and Douglas Fir, glued by Type 'A' phenol formaldehyde adhesive. The grain direction is oriented in the longitudinal direction to maximize strength and stiffness in the span direction (NelsonPine LVL Intro NZ, 2009). Strength values for the physical properties of NelsonPine LVL are presented in Table 4-1 (NelsonPine LVL General Information Brochure, 2008).

Property	Symbol	Characteristic Value
Modulus of Elasticity	E	10.7 GPa
Bending	f'_b	48.0 MPa
Tension Parallel to grain	f'_t	30.0 MPa
Compression Perpendicular to grain	f'_p	12.0 MPa

Compression Parallel to grain	f'_c	45.0 MPa
Shear in beam	f'_s	6.0 MPa
Density	ρ	570 kg/m^3

Table 4-1 – NelsonPine LVL Physical Properties

Testing specimens were constructed of 63mm x 150mm lengths of NelsonPine LVL with the characteristic properties as shown. LVL specimens were cut from as few board lengths as possible to reduce variability and provide maximum wood uniformity.

4.2. High Strength Steel Rods

Epoxy grouted test specimens used high strength steel threaded rods as part of the steel to wood connection. M16 Grade 8.8 high strength steel (as seen in Figure 4-2) was selected over mild steel for greater strength to prevent yielding, deformation, or failure in the steel rod.

Grade 8.8 steel threaded rods have a considerably higher yield stress of 680MPa, compared to a 300MPa yield stress for mild steel. The steel yield strength can be multiplied by the area of steel to obtain a theoretical tensile demand of 137kN required to yield the steel. This exceeds the expected capacity of the connections, to prevent a steel rod failure.

Figure 4-2 – Grade 8.8 Steel Threaded Rod

M16 Gr. 8.8 steel threaded rods were purchased in 1m sections from Blacks Fasteners of Christchurch. All rods were zinc plated to prevent corrosion and no additional surface preparation was engaged to affect the mechanical bond between the epoxy and threaded rod.

4.3. Epoxy Adhesives

Three epoxy adhesives were selected for testing the epoxy adhesive grouted steel threaded rods. These include the Fischer 'FIS V 360 S' Injection Mortar, JB Weld 'Industro Weld', and West 'Z206' Epoxy Hardener. Each product has been designated as a high temperature epoxy adhesive and designed to maintain strength at high temperatures.

4.3.1. Fischer 'FIS V 360 S' Injection Mortar

Designed as an injection mortar for adhering steel threaded rods into reinforced concrete, the Fischer 'FIS V 360 S' epoxy adhesive has been expanded to connect steel threaded rod into LVL sections as part of the steel to wood connection. The Fischer Injection Mortar consists of a two part mixture containing styrene-free vinylester resin with quartz sand and hardener. Fischer epoxy is separated by a two-chamber cartridge upon delivery but combined in a static mixer prior to use (IBMB, 2006). The Fischer epoxy, injection cartridge and static mixer can be seen in Figure 4-3:

Figure 4-3 – Fischer 'FIS V 360 S', 'AK' and Static Mixer

After the epoxy resin and epoxy hardener are combined in the static mixer, the epoxy adhesive has a working time of five minutes and a cure time at ambient temperature of 24 hours before full strength is achieved. According to the Fischer product catalogue, a fire resistance classification of F120 has been designated for epoxy adhesive use with steel threaded rods and reinforced concrete. This means Fischer epoxy embedded in concrete has been classified to resist design loads under fire conditions for up to 120 minutes.

Technical data for use of the Injection Mortar prescribes a minimum 125mm embedment and an 18mm diameter hole for use of a 16mm diameter steel threaded rod (FIS V 360 Product Catalogue, 2006). These values are well within the experimental criteria for testing specimens.

4.3.2. JB Weld 'Industro Weld'

The JB Weld 'Industro Weld' is described as an all-purpose cold-weld compound, allowing for curing at ambient temperature. Advertised for use with any porous or non-porous material, the 'Industro Weld' epoxy adhesive is suitable for the steel to wood connection experimentation. Consisting of a two part system, an epoxy-resin and a hardener, the 'Industro Weld' requires thorough hand mixing with a 1:1 mix ratio when combined from the two separate tubes prior to use. The JB Weld 'Industro Weld' epoxy is shown in Figure 4-4:

Figure 4-4 – JB Weld 'Industro Weld' Steel and Hardener

Following hand mixing, the product was injected using an empty cartridge and caulking gun. JB Weld epoxy had a working time of up to 30 minutes and a cure time of 24 hours. Product information claims 'Industro Weld' is resistant to temperatures up to 260°C, retaining strength far beyond the 100°C limit evidenced by previous testing (Barber, 1994). Product information does not list any other design requirements or restrictions for use, claiming its ubiquitous nature not only as an adhesive, but also a laminate, plug, filler sealant and electrical insulator noting (J-B Weld Product Description, 2009).

4.3.3. West System 'Z206' Epoxy Hardener and Adhesive Technologies 'ADR 310' Epoxy Resin

The West 'Z206' Epoxy Hardener was designed as a stud bonding adhesive intended for connecting timber, composite, concrete and other similar materials. Combined with the Adhesive Technologies 'ADR 310' Epoxy Resin, the 'Z206'-'ADR310' combination of epoxy adhesives (referred to herein as 'West') was developed as a high temperature post-cured carbon laminate to produce stronger and stiffer laminates with longer working times and a reliable cure time (Adhesive Technologies ADR Series Overview, 2009).

The two-part epoxy adhesive requires thorough hand mixing prior to use with a mix ratio of 13:1, 'ADR 310' to 'Z206'. The products are available in containers corresponding to the appropriate mix ratio. The smaller 'Z206' epoxy hardener is mixed in full with the 'ADR 310' epoxy resin. Both products are shown in Figure 4-5. When combined this mixture provides a 20 minute working time and cures at ambient temperature to maximum strength in 24 to 48 hours.

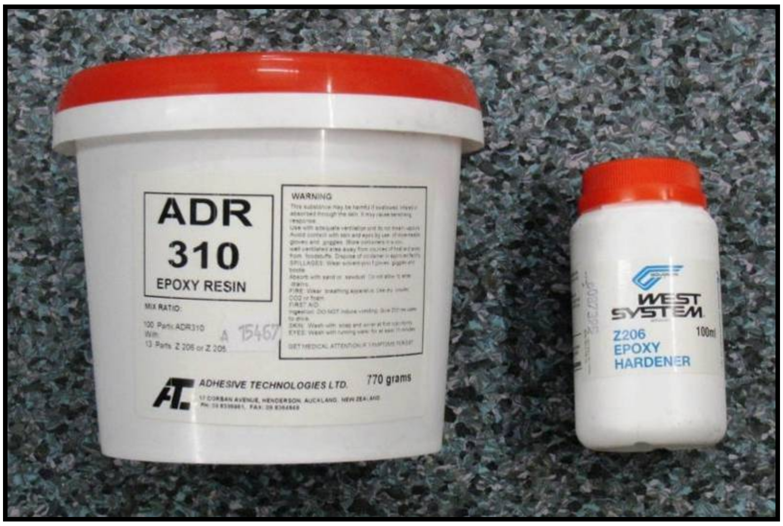

Figure 4-5 – West System ‘Z206’ Epoxy Hardener and Adhesive Technologies ‘ADR 310’ Epoxy Resin

4.4. Proprietary Mechanical Fasteners

Two proprietary mechanical fasteners have been selected for testing as alternatives to the epoxy grouted steel threaded rod connection. Proprietary mechanical fasteners include the Timberlinx ‘A475’ steel to wood connector and Lagscrewbolt connector. Both connections utilise mechanical bonds as opposed to adhesive bonds between the steel and wood materials.

4.4.1. Timberlinx ‘A475’

Timberlinx (pictured in Figure 4-6) is an embedded, two-part steel connection device used to mechanically fasten steel threaded rod to heavy timber. An expanding anchor bolt is placed within a hollow steel connector tube to simulate a mortise and tenon joint typically used in wood to wood connections. As an embedded hollow steel connection, Timberlinx provides the strength and stiffness of a stainless steel wood connector yet the aesthetic appeal of a characteristic wood connection (Moses, 2007).

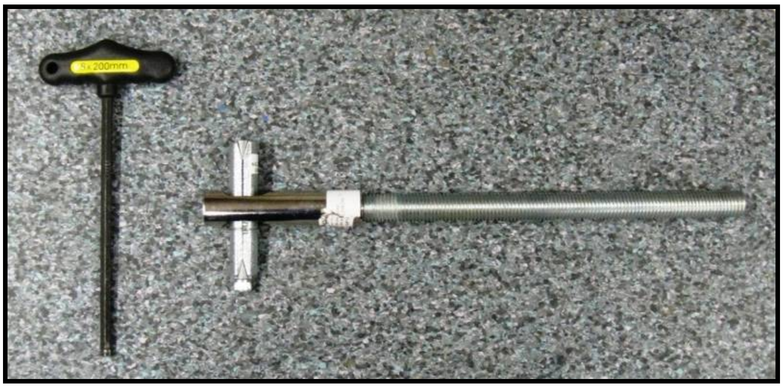

Figure 4-6 – Timberlinx 'A0475' with Expanding Anchor

4.4.2. Lagscrewbolt

The Lagscrewbolt connector (shown in Figure 4-7) presents an alternative solution for steel to wood connections in timber construction. Designed as a large lagscrew with threads all-around, Lagscrewbolt can be mechanically fastened much like a typical screw into wood. Lagscrewbolt, however, has a thread, similar to a nut, on the inside of the exposed end of the screw. This allows for the installation of steel threaded rods into the partially embedded Lagscrewbolt product (Nakatani, 2008).

Figure 4-7 – Lagscrewbolt

4.5. Preparation of Test Specimens

Test specimen construction occurred within the University of Canterbury Civil Engineering Laboratory using wood and steel machinery for specimen assembly. Test specimens for all three phases of testing were constructed from 63mm x 150mm NelsonPine LVL. Epoxy test specimens used M16 Grade 8.8 steel threaded rod and proprietary mechanical fasteners utilised steel mechanical connections.

4.5.1. Epoxy Test Specimens

Epoxy adhesive test specimens were constructed using the same method for all three epoxy products. Specimen preparation required cutting LVL lengths to size and injecting the epoxy adhesive and inserting the steel threaded rod to complete the steel to wood connection.

4.5.1.1. Cut LVL to Size

NelsonPine LVL arrived in 4m sections from the production plant. A drop table saw was used for cutting the specimens to length.

4.5.1.2. Drill Bracket Holes

An upright drill press in the Civil Engineering Laboratory was used to drill bracket holes in test specimens. Specimens were clamped to the drill table and holes for each of the four 16mm diameter bolts were drilled.

4.5.1.3. Cut Steel Threaded Rod to Size

M16 Grade 8.8 steel threaded rod was attained in 1m lengths and cut to size in the Civil Engineering Laboratory with the wet cut metal saw. A tolerance of 5mm was used to establish minimum and maximum lengths for steel threaded rod.

4.5.1.4. Drill Grout Holes

A radial arm drill press was used to drill holes for the epoxy adhesive grout. The 18mm diameter drill bit was marked at 150mm embedment from the tip of the bit and drilled perpendicular into the wood at a constant, steady rate. After drilling, excess wood particles were cleared from the drill hole by a pneumatic air tube inserted to the bottom of the 150mm deep hole to maintain a clean bonding surface.

4.5.1.5. Level Specimens

Test specimens were arranged between two pieces of LVL timber and clamped by large C-clamps to prevent movement during preparation. A level (seen in Figure 4-8) was used to straighten each test specimen in preparation for injection of the epoxy adhesive grout and insertion of the steel threaded rods.

Figure 4-8 – Levelling Epoxy Test Specimens

4.5.1.6. Mix Epoxy

While the Fischer epoxy came with a static mixing tube that automatically mixed the epoxy resin and epoxy hardener, both the JB Weld and West products required thorough hand. Epoxy resins and epoxy hardeners were mixed to specified ratios according to weight and measured on a digital scale to confirm an exact mix ratio.

4.5.1.7. Pour Epoxy

Epoxy was injected into each of the grout holes through the use of a caulking gun and cartridge system. A 200mm nozzle was extended to the bottom of the grout hole (as seen in Figure 4-9) for injection and epoxy was injected 2/3 of the way up the embedment length, as instructed in product catalogues (FIS V 360 Product Catalogue, 2006).

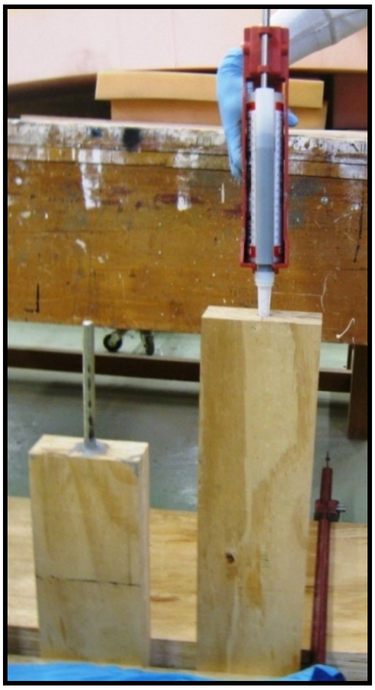

Figure 4-9 – Pouring Epoxy Adhesive to Bottom of Grout Embedment Hole

4.5.1.8. Fill Threads with Epoxy and Insert into Grout Hole

Experiments with trial specimens tested prior to actual testing revealed a problem with epoxy penetration between the threads on steel threaded rods. To alleviate this issue, the full embedment length on the steel rod was marked and epoxy was applied by hand to fill any gaps in the steel threads prior to insertion into the grout hole (as seen in Figure 4-10). This procedure was completed within the specified curing time for each of the epoxy types.

Figure 4-10 – Epoxy-Filled Steel Threads

4.5.1.9. Level Steel Threaded Rods

Once the steel threaded rods were inserted into the epoxy-grouted LVL test specimens, a level was used to make sure each rod was inserted perpendicular to the face of the LVL specimen. Steel threaded rods were centred within the working time allotted for each epoxy product. A level was used to check both the longitudinal and perpendicular directions to make sure the steel rod was protruding from the grout hole at a right angle to prevent any eccentricity and design for pure axial tension.

C-clamps were left in place and the level was used to confirm that all specimens were perfectly upright and steel threaded rods were protruding at right angles to the face of the LVL. Epoxy grouted test specimens were allowed to cure (as shown in Figure 4-11) in the Civil Engineering Laboratory Wood-Working Room at ambient temperature and humidity for a minimum of 24 hours prior to testing.

Figure 4-11 – Epoxy Test Specimen Curing

4.5.2. Timberlinx Test Specimens

Timberlinx test specimens were constructed similar to the epoxy test specimens, with slight modifications to accommodate a different connection type.

4.5.2.1. LVL Preparation

LVL sections were cut using the wood drop table saw and were constrained to the same tolerance for acceptance as for epoxy specimens. Bracket holes were drilled using the upright drill press. The primary difference occurred after cutting the LVL and drilling the bracket holes, as different sizes and locations of holes were used for the Timberlinx product.

4.5.2.2. Drill Central Hollow Steel Rod Hole

The radial arm drill press was used to drill a 22mm diameter central hole into the face of the LVL section using a custom drill bit included as part of the Timberlinx product package.

4.5.2.3. Drill Expanding Anchor Hole

LVL test specimens were repositioned on the radial arm drill press table to drill an expanding anchor hole perpendicular to the original central steel rod hole.

4.5.2.4. Tighten Expanding Anchor

The final step in the Timberlinx test specimen assembly was to hand tighten the expanding anchors. After confirming that the anchor fit firmly between the end of the wood and the central hollow steel tube slot, the anchor was hand tightened as tight as possible. Specimens were checked to verify the Timberlinx 'A475' was firmly embedded in the wood and flush with the face of the LVL. Hand-tightening is shown in Figure 4-12:

Figure 4-12 – Expanding Anchor Tightening

4.5.3. Lagscrewbolt Test Specimens

Lagscrewbolt test specimens were constructed using the same basic procedure as epoxy grouted test specimens.

4.5.3.1. LVL Preparation

LVL sections were cut using the wood drop table saw and were constrained to the same tolerance for acceptance as for epoxy. Bracket holes were drilled using the upright drill press. The primary difference occurred after cutting the LVL and drilling the bracket holes, as an additional pilot hole was required for insertion of the Lagscrewbolt specimen.

4.5.3.2. Drill Pilot Hole for Lagscrewbolt

Lagscrewbolt literature prescribed the drilling of a pilot hole centred at the face of the LVL section (Nakatani, 2009). The LVL section was clamped to the side of the radial arm drill press to drill a pilot hole into the test specimen. The 22mm diameter drill bit was marked at 200mm, per the embedment length, and drilled perpendicular into the wood at a constant, steady rate. Excess wood particles were cleared from the pilot hole by a pneumatic air tube inserted to the bottom of the 200mm deep hole.

4.5.3.3. Insert Lagscrewbolt with Hand Wrench

A hand wrench was used to insert the Lagscrewbolt threaded screw into the pilot hole. Specimens were hand tightened until all screw threads were embedded into the wood, leaving the threaded rod shank at the free end exposed. This allowed for quick and easy insertion of a 12mm diameter Grade 8.8 high strength threaded rod at the free end, completing the steel to wood connection. Testing Equipment

Experimental testing used machinery in the University of Canterbury Civil Engineering and Chemical and Process Engineering Laboratories. Tensile pull out tests utilised the Avery Testing Machine for cold, oven and cooled tests. The custom furnace was used for furnace testing. A custom steel bracket was constructed for holding the testing specimens in place at the fixed end for testing in both the Avery Testing Machine and custom furnace.

4.6.1. Avery Testing Machine

The Avery Testing Machine, located in the University of Canterbury Civil Engineering Laboratory, is a two-part testing machine used to apply incremental increasing load to testing specimens. A loading dock (shown left in Figure 4-13) with threaded clamps at the top and bottom of the dock, works in tandem with a numerical display (shown right in Figure 4-13) to apply an increasing tensile force to specimens.

Figure 4-13 – Avery Testing Machine

Three of the four phases of testing were conducted with the Avery Testing Machine. These included cold testing, oven and cooled testing of specimens. With a maximum tensile load of 500kN, the Avery Testing Machine capably applied sufficient load to force a connection failure in all test specimens.

4.6.2. Custom Furnace

The custom furnace in the University of Canterbury Chemical and Process Engineering Laboratory was used for furnace testing of steel to wood test specimens. The custom furnace is composed of three parts. A computer (at centre in Figure 4-14) supported the Universal Data Logging (UDL) software for recording testing data. A hydraulic ram with an attached load-cell (at right in Figure 4-14) maintained a constant tensile load on test specimens. The custom furnace (at left in Figure 4-14) was used for heat application. The front face of the furnace provided a digital thermometer display that was used to specify a given temperature, up to 850°C. The digital thermometer also displayed the current temperature within the furnace during testing.

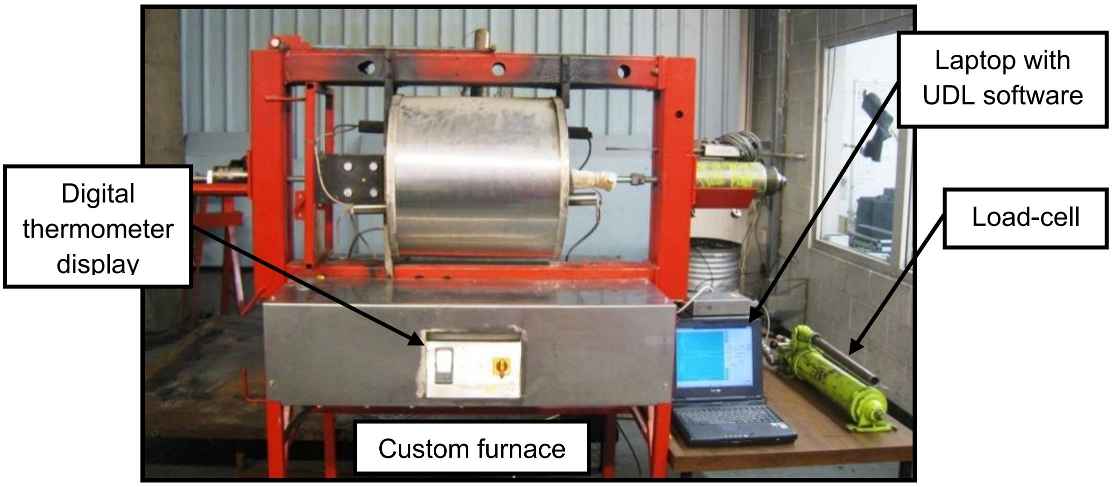

Figure 4-14 – Custom Furnace

4.6.3. Custom Bracket

A custom bracket (shown in Figure 4-15 and Figure 4-16) was constructed to restrain one end of the steel to wood connection. Steel plates and bolts were oversized to prevent a connection failure at the custom bracket end of the test specimen. Tensile testing demonstrated connection failures at the experimental end of the steel to wood connection, opposite the bracket end, as intended.

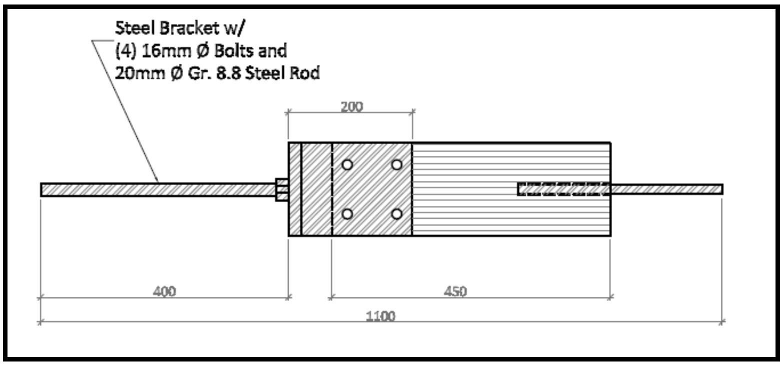

Figure 4-15 – Custom Bracket Schematic Drawing

Figure 4-16 – Custom Bracket

4.6. Calibration

It was critical to maintain and calibrate all testing equipment. This confirmed load measurements used for data analysis were accurate. Only by maintaining calibrated equipment can results be used with confidence.

4.7.1. Avery Testing Machine

Experimental data recording on the Avery Testing Machine was conducted using two methods. The numerical display (seen at right in Figure 4-13) on the Avery Testing Machine provided a visual display for the applied load. The Universal Data Logging (UDL) interface (shown in Figure 4-17) supported by the Civil Engineering Laboratory computer provided a digital display for the applied load. The computer was attached to the Avery Testing Machine using load cells to digitally record the applied force. While the digital UDL software was the primary recording mechanism, the numerical display provided load values that were used to check accuracy with the digital readings.

The calibration feature within the UDL software package provided a concise method for maintaining accuracy of digital readings. This was accomplished by programming minimum and maximum values within the software, allowing for a linear calculation of applied loads.

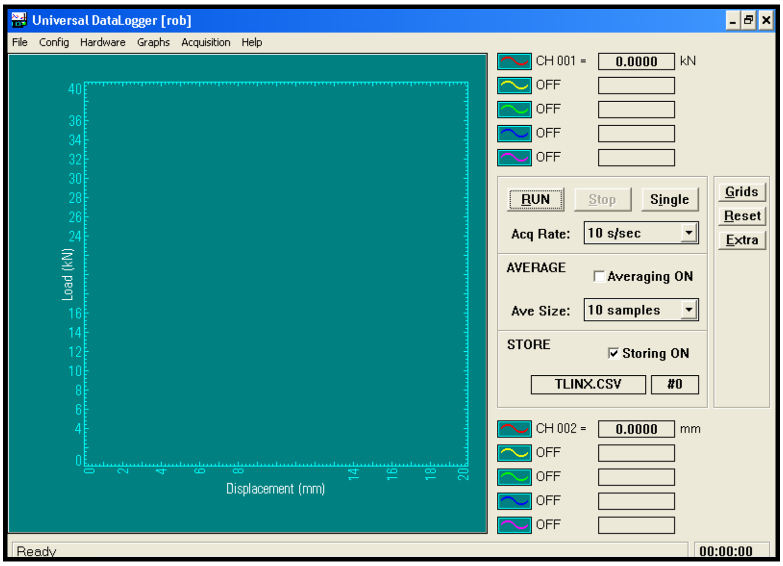

Figure 4-17 – UDL Software Interface

To calibrate for applied load, a sample test specimen was used in the Avery Testing Machine. Before engaging the testing specimen, the numerical display confirmed a 0kN load, which was programmed as the minimum value within the UDL software. The test specimen was then loaded to a value of 150kN, confirmed by the Avery numerical display, and programmed as a maximum in the UDL software. By calibrating minimum and maximum values, the UDL digital readings would be accurate for any applied load between 0kN and 150kN.

4.7.2. Custom Furnace

Several items were used to record data for use with the custom furnace. These included a load cell attached to the hydraulic ram used for furnace testing, and a thermocouple located in the centre of the custom furnace flue. Each of these items was supported by UDL software to record the applied load and temperature over time.

The load cell was calibrated in the same fashion as the cell used with the Avery Testing Machine. The load cell was installed into the Avery Testing Machine and subjected to loads of 0kN and 150kN. Establishing minimum and maximum values allowed for linear interpolation of applied loads between these values. The thermocouple and digital thermometer provided with the custom furnace required no additional calibration for use.

5. Cold Testing

The first phase of experimentation, testing steel to wood connections at ambient conditions, was performed to determine the maximum ultimate strength at cold temperatures. Ultimate strengths from cold testing were established as the control values for testing and used as the basis for comparison for oven and cooled test results.

5.1. Cold Test Specimens

Cold test specimens were composed of LVL and high strength steel threaded rod. High temperature epoxy was used to connect the steel threaded rod to the LVL. Proprietary mechanical fasteners Timberlinx and Lagscrewbolt were used to connect the steel threaded rod to the wood using a mechanical connection as opposed to an adhesive connection used by the epoxy grout.

5.1.1. Epoxy Specimens

Epoxy test specimens were prepared in accordance with the protocol discussed in Section 4.5.1. A schematic drawing for epoxy test specimens is shown in Figure 5-1, with the specimen displayed in Figure 5-2:

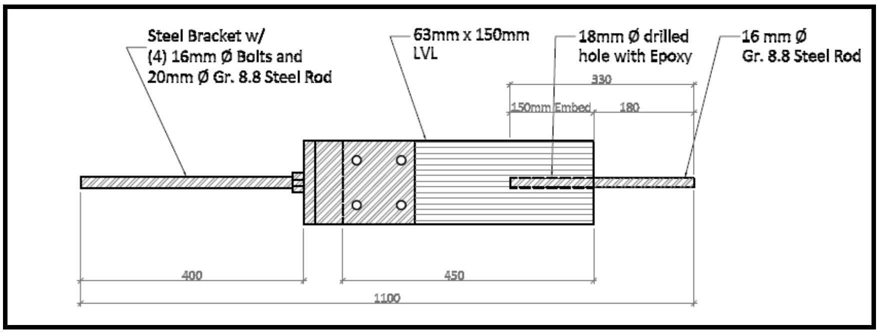

Figure 5-1 – Epoxy Cold Test Specimen Schematic Drawing

All epoxy adhesive cold test specimens consisted of a 450mm section of NelsonPine LVL. A 16mm diameter Grade 8.8 high strength steel threaded rod was epoxy grouted into an 18mm diameter central drill hole. The custom steel bracket was bolted onto test specimens for testing in the Avery Testing Machine. The only variation in epoxy test specimens occurred in the type of epoxy adhesive used for grouting the steel threaded rod into the LVL section. The Fischer 'FIS V 360 S', JB Weld 'Industro Weld', and West 'Z206' high temperature epoxies were used in testing.

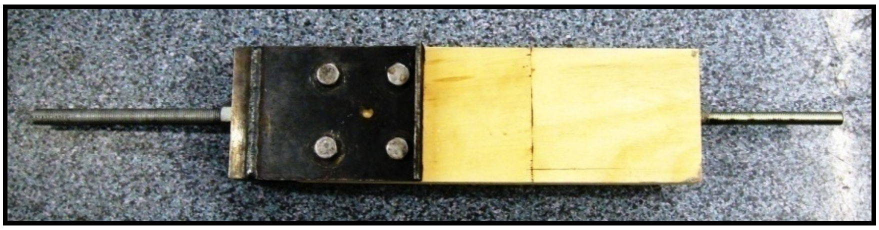

Figure 5-2 – Epoxy Cold Test Specimen

5.1.2. Timberlinx Specimens

Timberlinx test specimens were constructed according to the procedure presented in Section 4.5.2. Specimens consisted of a 450mm length NelsonPine LVL section with holes drilled for the custom drill bit and the 'A475' hollow steel tube and expanding anchor, as shown in Figure 5-3 and Figure 5-4.

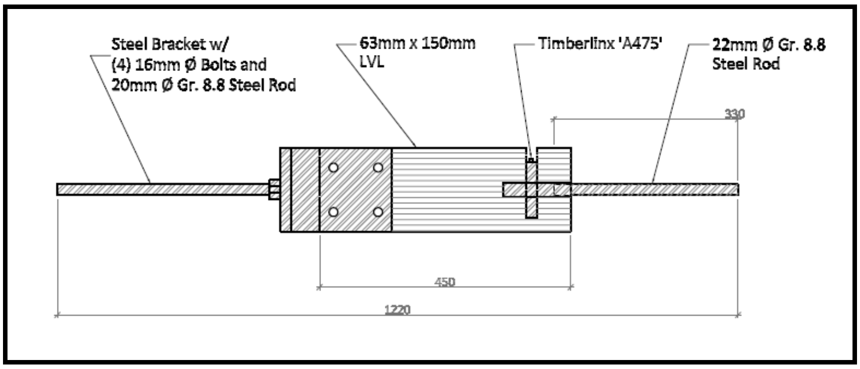

Figure 5-3 – Timberlinx Cold Test Specimen Schematic Drawing

The custom steel bracket was bolted to Timberlinx test specimens for testing. A 7/8" diameter Grade 8.8 high strength steel threaded rod was inserted into the free end of the 'A475' product to provide a grip at the connection end. Both the steel bracket and rod provided grips for the Avery Testing Machine to apply the tensile load for experimentation.

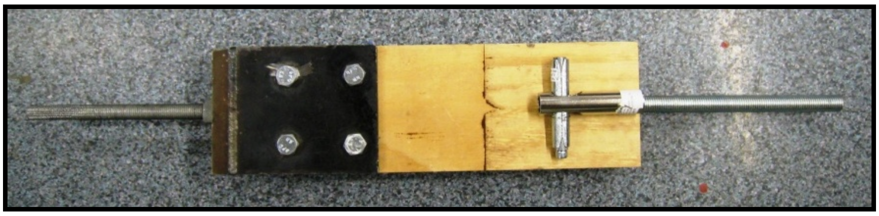

Figure 5-4 – Timberlinx Cold Test Specimen

5.1.3. Lagscrewbolt Specimens

Tests specimens using the Lagscrewbolt threaded screw connection were constructed according to the procedure established in Section 4.5.3. Lagscrewbolt test specimens were constructed with 450mm length NelsonPine LVL. The Lagscrewbolt product was hand screwed into the 22mm diameter pilot hole and a 12mm diameter Grade 8.8 high strength steel threaded rod was screwed into the connection end of the product. The custom steel bracket was bolted to the opposite end for testing in the Avery Testing Machine. A Lagscrewbolt test specimen is shown in Figure 5-5 and Figure 5-6:

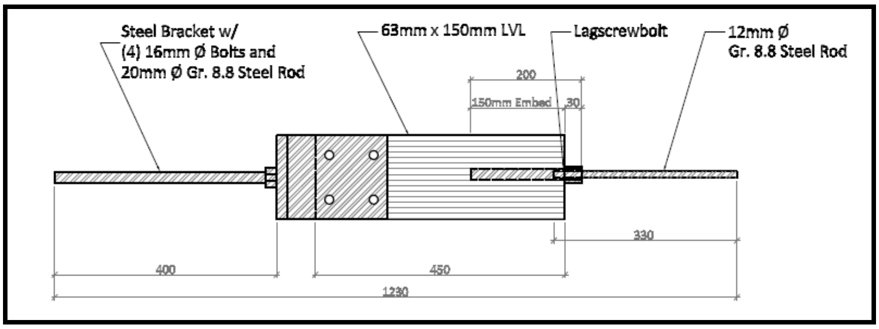

Figure 5-5 – Lagscrewbolt Cold Test Specimen Schematic Drawing

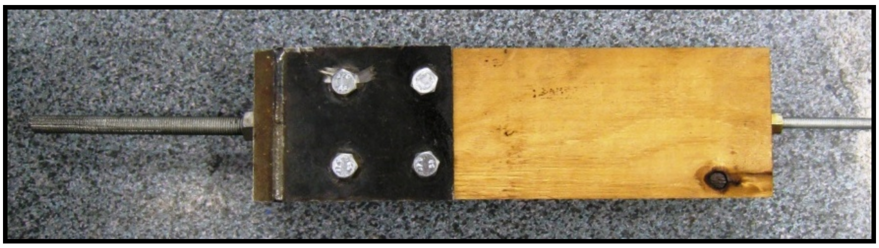

Figure 5-6 – Lagscrewbolt Cold Test Specimen

5.2. Testing Procedure

Cold testing was performed in the University of Canterbury Civil Engineering Laboratory with the Avery Testing Machine to evaluate the ultimate strength of steel to wood connections at ambient conditions.

5.2.1. Specimen Preparation

Prior to actual cold testing, completed specimens required additional preparation. High strength bolts on the custom steel bracket were hand tightened to provide a firm grip at the restrained end of the specimens for placement in the Avery. The bracket was hand-tightened force a failure at the experimental end of the connection.

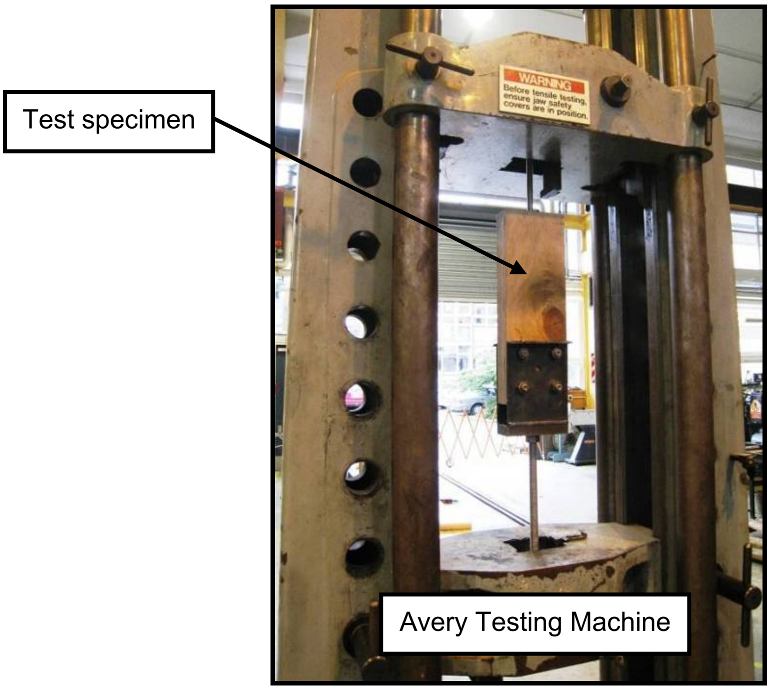

Figure 5-7 – Cold Test Specimen with Custom Bracket in the Avery Testing Machine

Once the test specimens were fitted with a custom bracket, samples were loaded into the Avery Testing Machine, as shown in Figure 5-8. After verifying the digital readings from the load cell (seen on the computer at right in Figure 5-8) corresponded to the readings on the Avery Testing Machine (shown at centre in Figure 5-8), the cold tests were ready to commence.

Figure 5-8 – Avery Testing Machine Specimen Set Up

5.2.2. Load Application

Tensile load for cold testing was applied at a constant rate of application using the numerical display on the face of the Avery Testing Machine. This numerical display, as seen in the centre of Figure 5-8, displays the load, in kN, applied to test specimens. The display dictates the rate at which the tensile load is applied to Avery Testing Machine test specimens. A constant rate of tensile load application of approximately 10kN per minute was applied to specimens during testing. This rate was slow enough to prevent a sudden impact loading, yet fast enough to perform tests in a reasonable amount of time. The tensile load was increased at a constant rate of deformation until failure, at which point the load was removed.

5.2.3. Data Recording

Ultimate strength values were digitally recorded as a function of time using the Universal Data Logging (UDL) software programmed on the University of Canterbury Civil Engineering Laboratory Computer. Load values were tracked by load cells installed within the Avery Testing Machine.

To verify the load values, the numerical display, shown at centre in Figure 5-8, was checked to compare actual load values with digital load values within the UDL recording software. Upon failure, a final confirmation for ultimate load on the numerical display was checked with UDL readings for acceptance of testing data.

5.2.4. Specimen Failure

Connection failure was evaluated purely in the strength domain. Failure occurred when the connection could no longer sustain any additional load.

Defining failure to be solely in the strength domain provided the most concrete method for defining failure and was based off of previous testing with steel threaded rods embedded in LVL (Barber, 1994 and Harris, 2004). Once failure occurred and was confirmed by the digital readings, the test was completed and data recording was discontinued.

5.3. Results

Cold testing was performed to determine the ultimate strength for steel to wood connections at ambient conditions. Ultimate loads found through cold testing were established as the control values for each connection.

Results for ultimate loads and failure modes established through cold testing can be found in Table 5-1 with a plot found in Figure 5-9:

Specimen	Embedment Length (mm)	Hole Diameter (mm)	Ultimate Load (kN)	Failure Mode
Fischer	150	18	60.3	1
JB Weld	150	18	66.8	3
West	150	18	74.1	3
Timberlinx	-	-	36.0	2
Lagscrewbolt	-	-	42.7	1

Table 5-1 – Cold Test Results

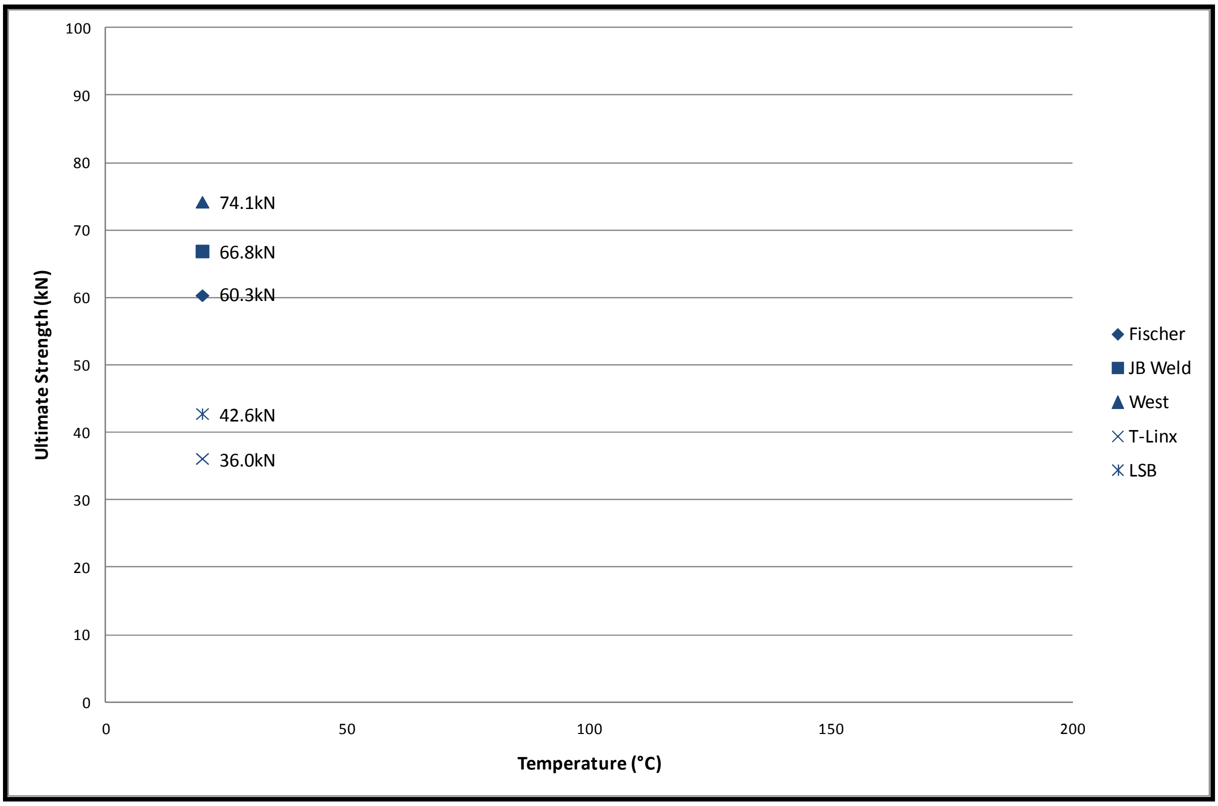

Figure 5-9 – Cold Test Results

Testing data indicates epoxy grouted steel to wood connections provide greater ultimate load values than proprietary mechanical fasteners at ambient conditions. Ultimate strength values for the three epoxy grouted steel to wood specimens range from approximately 58kN to 75kN, with slightly lower values from the Timberlinx and Lagscrewbolt proprietary mechanical fasteners between 36kN and 43kN.

While each of the three epoxy specimens differs in consistency and contents, all three demonstrate considerable strength at ambient conditions. Both the JB Weld and West products demonstrate ultimate strength around 70kN while the Fischer epoxy displays a lower ultimate strength around 60kN.

As steel proprietary mechanical fasteners, both Timberlinx and Lagscrewbolt rely on LVL strength to achieve steel to wood connection performance, as the steel was not expected to be a possible failure mechanism. Larger sections of LVL could likely achieve greater performance.

5.4. Failure Modes

Results from cold testing revealed three distinct failure modes for the epoxy and proprietary steel to wood connections. Each of the three failure modes exhibited a brittle failure occurring due to the strength of the wood, demonstrating the considerable strength of the steel to wood connection at ambient conditions.

5.4.1. Mode 1 Failure

Failure Mode 1 occurred when the wood split perpendicular to the laminations. As tensile load increased, hairline cracks began to develop until failure occurred with an abrupt break at the original crack. This wood failure occurring perpendicular to the laminations in the weak axis suggests that the strength of the connection was sufficient to enable a splitting force in the LVL as the dominant failure mechanism. Mode 1 Failures, as shown in Figure 5-10, can be characterized as confinement failures in the wood, as a larger cross section of LVL could mitigate this failure mechanism.

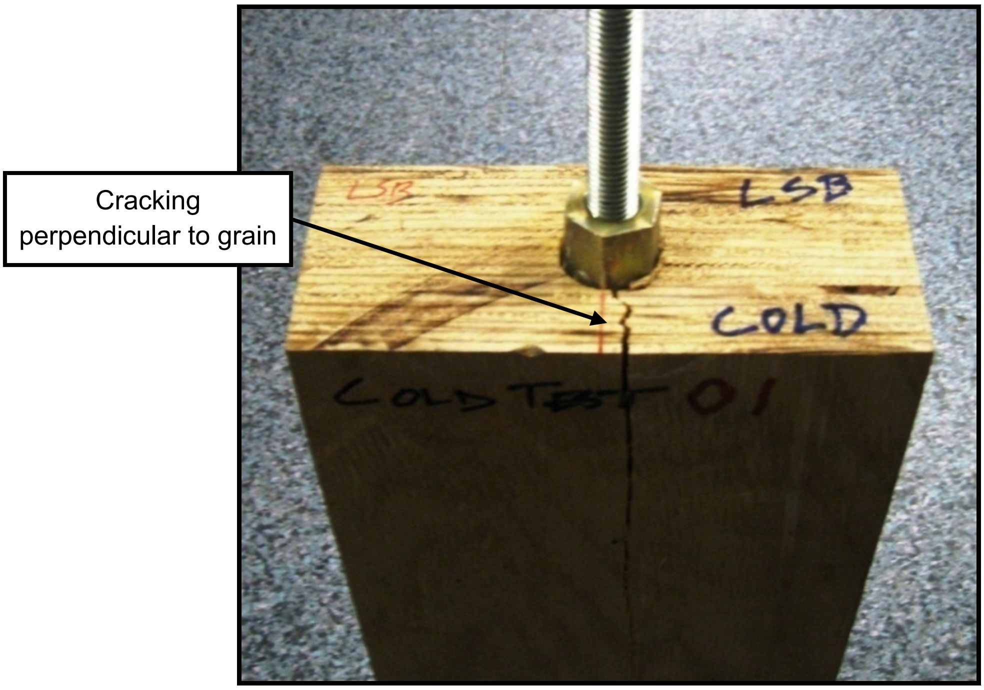

Figure 5-10 – Mode 1 Failure in Lagscrewbolt Specimen

Splitting perpendicular to the wood laminations was also observed in testing conducted by van Houtte (2003) in which cracks began to occur at the minimum edge distance within LVL specimens. For the 63mm x 150mm specimens tested, the critical edge distance occurred perpendicular to the laminations, supporting van Houtte's (2003) observations.

A Mode 1 Failure occurring in the LVL section is a primary indicator of several main conclusions. First, the steel to wood connection maintained sufficient bonding to the wood to prevent a pull out failure from occurring. Second, the stresses parallel to the laminations were insufficient to cause splitting in the strong axis direction. Finally, the minimum edge distance coincided with the weak axis direction as the failure location. As tensile stresses in the connection increased, shear stresses in the wood caused splitting in line with the connection and ultimately caused the confinement failure.

5.4.2. Mode 2 Failure

A Mode 2 failure was defined as a wood failure occurring parallel to the LVL laminations. This brittle failure mechanism was observed in Timberlinx sections only, as this proprietary steel to wood connection is unique in design and function compared to the epoxy grout and Lagscrewbolt connections. The expanding anchor creates a mechanical bond perpendicular to the wood, unlike the mechanical bond created by threads on the Lagscrewbolt specimen and the adhesive bond with epoxy steel threaded rods. A Mode 2 Failure for a Timberlinx cold test specimen is shown in Figure 5-11:

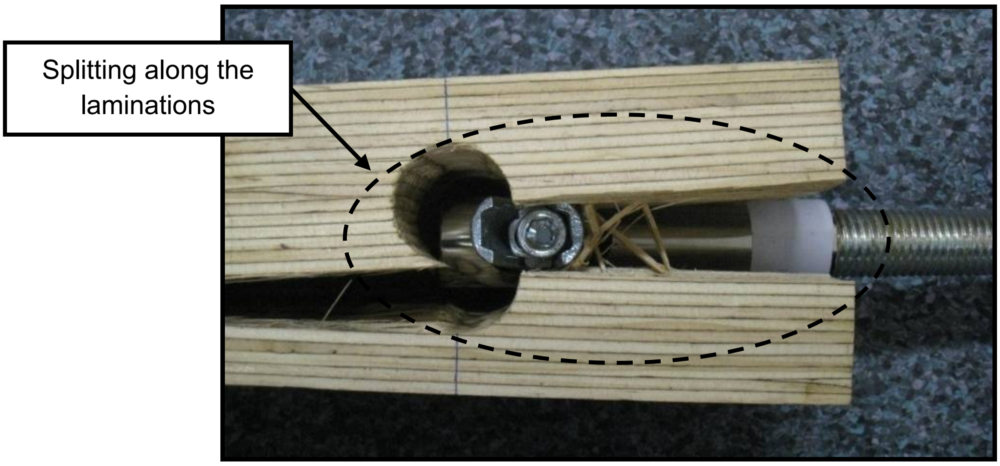

Figure 5-11 – Mode 2 Failure in Timberlinx Specimen

As the tensile force increased, the expanding anchor inserted perpendicular to the laminations induced compression on the restraining wood section and caused splitting adjacent to the bar, parallel to the laminations. This failure mode with splitting in line with the expanding anchor was expected for Timberlinx connections, as pull out of steel threaded rod and the previously observed Mode 1 failure with splitting perpendicular to the laminations were both unlikely to occur.

The observed Mode 2 Failure in the Timberlinx product during cold testing resulted in a wood failure occurring in the LVL. Significantly stronger and more ductile, the Timberlinx steel product would require significantly greater load to cause deformation or failure in the steel.

5.4.3. Mode 3 Failure

A Mode 3 Failure occurred when the epoxy grouted steel threaded rod and a significant amount of timber pulled out from the test specimen. Occurring in both the JB Weld and West epoxies, a Mode 3 Failure was classified as a wood failure at the epoxy-wood interface. As tensile stresses in the testing specimen increased, shear stresses along the epoxy-wood interface transferring the force from the rod to the wood increased, causing pull out along the interface. Having a significant amount of wood attached to the epoxy grouted steel rod identified the failure as a wood failure as opposed to an epoxy failure. Failure Mode 3 is shown in Figure 5-12:

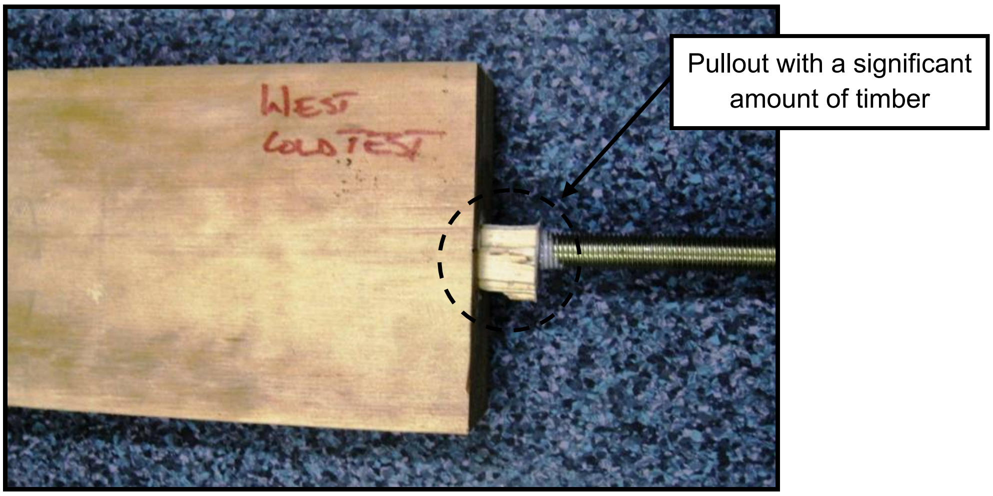

Figure 5-12 – Mode 3 Failure in West Specimen

5.5. Comparison with Previous Testing

It is important to verify testing results for cold test data by comparing ultimate load values against previously observed behaviour. Resources including the Timber Design Guide, previous testing for epoxy grouted steel rod connections, and independent testing for proprietary mechanical fasteners provide equations and analysis that can be used for calculating the expected failure loads and assessing cold test results.

5.5.1. Epoxy Specimens

Several resources can be consulted to evaluate the cold test results against previously recorded data for epoxy specimens. Previous work by van Houtte (2003) and Deng (1997) established the maximum pull out force for epoxy grouted steel threaded rod specimens. The Timber Design Guide (2007) provides equations for calculating potential failure modes for steel to wood connections.

5.5.1.1. Van Houtte (2003)

Tensile pull out tests of steel threaded rod in LVL were conducted by van Houtte (2003). Testing criteria evaluated a variety of variables including embedment length, edge distance, bar size and the addition of self-tapping screws for all-purpose epoxy. Ultimately, van Houtte (2003) produced a formula designed to predict the pull out capacity of epoxy grouted steel threaded rod connections. The pull out formula, based on extensive testing with epoxy grouted high strength steel threaded rods in 105mm x 105mm LVL specimens, is intended for embedment lengths ranging from 50mm to 400mm and is presented in Equation 5-1:

$$F = K\left[\left(1.885\times10^{-4}\right)E^2 L f_s + 15\right] \quad \boldsymbol{F = K\left\{\left(1.885 \times 10^{-4}\right) E^2 L f_s + 15\right\}}$$ **Equation 5-1**

F = Pull out force (kN)

K = 0.85 Reduction Factor (Assume 1.0 for calculating ultimate strength values)

E = Embedment hole diameter (mm)

L = Embedment length (mm)

f_s = LVL shear stress (MPa)

Inserting the given cold testing conditions for epoxy grouted steel threaded rods yields the following equation based on van Houtte's predicted pull out force:

$$F = 1.0\left[\left(1.885\times10^{-4}\right)(18mm)^2(150mm)(6.0MPa)+15\right]$$

$$F = 70.0kN$$

The predicted pull out force for tensile tests at ambient conditions is approximately 70kN for an 18mm diameter embedment hole, 150mm embedment and 6.0MPa shear stress for NelsonPine LVL. Experimental failure loads are presented with the predicted pull out load in Figure 5-13:

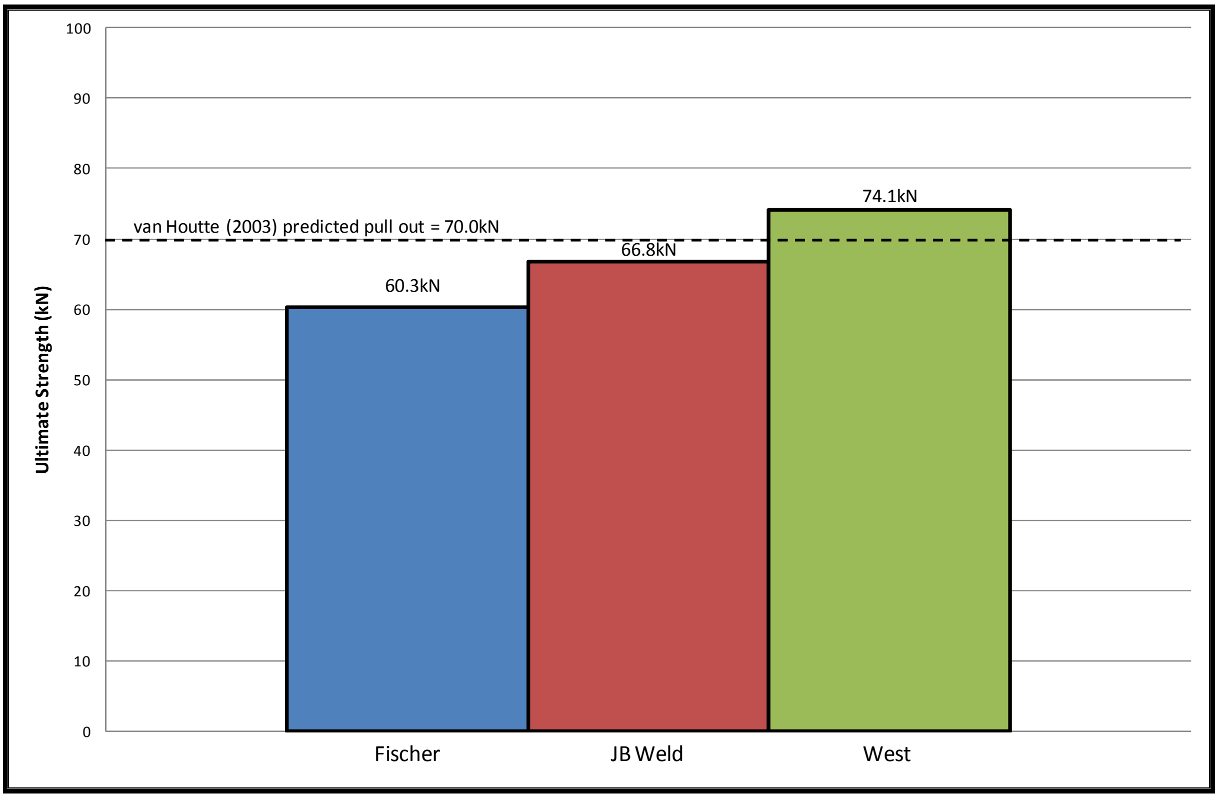

Figure 5-13 – Cold Test - van Houtte (2003) Comparison

An ultimate strength comparison for epoxy grouted steel threaded rods in LVL displays that both the JB Weld and West epoxy specimens compare favourably with the van Houtte pull out force. The ultimate strength with Fischer epoxy, however, falls short of the 70kN predicted pull out load. This is possibly due to the failure mechanism, as the Fischer epoxy displayed a Mode 1 Failure with splitting of the wood perpendicular to the laminations. Both the JB Weld and West specimens exhibited a Mode 3 Failure, pull out at the epoxy-wood interface.

5.5.1.2. Deng (1997)

A series of tensile tests were performed by Deng (1997) with epoxy grouted steel threaded rod in glulam timber as opposed to LVL. Tensile tests were performed using 105mm x 105mm LVL specimens with 16mm diameter steel threaded rod in 20mm diameter holes grouted with all-purpose epoxy. While this presents a slight variation from the LVL used in van Houtte (2003) and cold testing, glulam wood is very similar to LVL, with slightly better performance in tension in the direction of the laminations (van Houtte, 2003 and Harris, 2004). An experimental prediction for the ultimate load of epoxy grouted steel threaded rod in glulam timber is found in research by Deng (1997) and presented in Equation 5-2:

$$F = 10.94 k_b k_e k_m \left(\frac{l}{d}\right)^{0.86} \left(\frac{d}{20}\right)^{1.62} \left(\frac{h}{d}\right)^{0.5} \left(\frac{e}{d}\right)^{0.5}$$
$$F = 10.94 k_b k_e k_m \left(\frac{l}{d}\right)^{0.86} \left(\frac{d}{20}\right)^{1.62} \left(\frac{h}{d}\right)^{0.5} \left(\frac{e}{d}\right)^{0.5}$$

Equation 5-2

F = Ultimate tensile load of the connection (kN)

k_b k_b = Bar type factor (1.0 for steel threaded rod)

k_e k_e = Epoxy factor (Assume 1.0 for comparison)

k_m k_m = Moisture content factor (1.0 for < 15% moisture content)

l = Embedment length (mm) (5d ≤ *l* ≤ 15d)

d = Steel bar diameter (mm) ($16 \leq d \leq 24$)

h = Hole diameter (mm) ($1.15d \leq h \leq 1.4d$)

e = Edge distance from centre of steel bar (mm) ($1.5d \leq e \leq 3d$)

Inserting the cold testing parameters into the equation and assuming safety factors of 1.0 gives the following equation:

$$F = 10.94(1.0)(1.0)(1.0)\left(\frac{150mm}{16mm}\right)^{0.86}\left(\frac{16mm}{20}\right)^{1.62}\left(\frac{18mm}{16mm}\right)^{0.5}\left(\frac{31.5mm}{16mm}\right)^{0.5}$$

$$F = 77.7kN$$

The predicted ultimate tensile load from Deng (1997) suggests a maximum load of nearly 78kN for the steel to wood connection. Analysis assumed an embedment length of 150mm, bar diameter of 16mm, hole diameter of 18mm and edge distance of 31.5mm for a 63mm wide LVL specimen. The calculation also assumes all safety and reduction factors have been set to 1.0 to give an ultimate load. Experimental failure loads from cold testing are presented with the predicted ultimate failure loads in Figure 5-14:

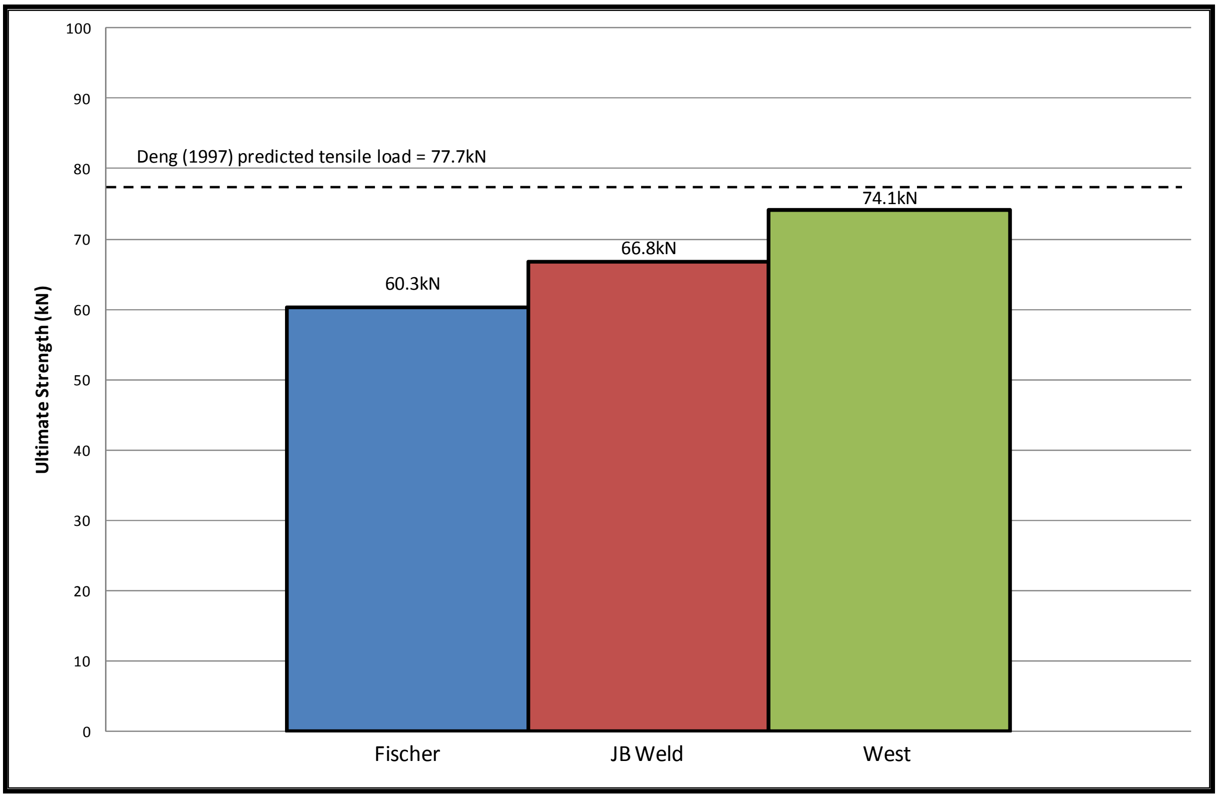

Figure 5-14 – Cold Test - Deng (1997) Comparison

Comparing the ultimate strengths of the epoxy grouted specimens to the predicted ultimate failure load indicates that all three specimens fall short of this value. Despite the equation predicting ultimate load as opposed to only pull out failure, as presented in van Houtte (2003), all three specimens fail to meet the predicted load. In addition, the Fischer epoxy ultimate strength appears substantially lower than the expected force value, possibly due to the difference in observed failure mechanism.

Variation between glulam and LVL samples could cause differences in the ultimate strength of epoxy samples. It has been observed in van Houtte (2003) and Harris (2004) that epoxy grouted steel threaded rod connections in glulam demonstrate greater strength values when compared to embedment in LVL. Additional research is necessary to quantify an appropriate reduction value for ultimate strength in LVL compared to glulam wood.

In addition to the difference in wood type, it should be noted that cold test specimens fall outside of the parameters for hole diameter (*h*). Deng (1997) suggests the equation for ultimate tensile load be used for hole diameters between 1.15d and 1.4d. This is equivalent to 18.4mm to 22.4mm for a 16mm diameter steel threaded rod. Despite the variation, it is expected that having an 18mm diameter hole as opposed to a 18.4mm diameter hole is negligible.

5.5.1.3. Timber Design Guide (2007)

The Timber Design Guide (2007) provides guidance for calculating the expected failure load for epoxy bonded steel connections subjected to axial tension. The procedure involves quantifying three failure mechanisms including a steel yielding failure, wood fracture failure and bar pull out failure for steel to wood connections. To determine ultimate strength loads for comparison with cold testing, all safety, strength and reduction factors have been assumed to be 1.0. For reference, the Timber Design Guide provides analysis techniques for steel threaded rod into glulam as opposed to LVL.

The first failure mechanism check included in the Timber Design Guide was a check for steel yielding. Wood design requires that the capacity of the steel threaded rod be greater than the demand. The capacity of the connection can be calculated using Equation 5-3:

$$(\varphi Q_n)_{steel} = \varphi_{steel} n A_s f_y \qquad \textbf{Equation 5-3}$$

φ φ = Strength reduction factor (from NZS 3603:1993, Assume 1.0 for ultimate strength)

Q_n Q_n = Nominal axial strength of the connection (kN)

φ_{steel} φ_{steel} = 0.8 (NZS 3404:1992 for steel members in tension, Assume 1.0 for ultimate strength)

n n = Number of steel bars

A_s A_s = Cross sectional area of each steel bar (mm^2/1000)

f_y f_y = Characteristic yield strength of steel (MPa)

Inserting the cold testing variables into the steel yielding capacity calculation gives the following equation:

$$(1.0Q_n)_{steel} = (1.0)(1.0)\left(\frac{\pi 8mm^2}{1000}\right)(680MPa)$$

$$(Q_n)_{steel} = 136.7kN$$

Based upon a 16mm diameter high strength steel threaded rod, an axial tensile load of nearly 137kN would be required to yield the steel. Considering the maximum ultimate load of 75kN observed in cold testing, this 137kN yield strength suggests that yielding of the steel was an unlikely failure mechanism.

The second failure mechanism to be checked using was a wood fracture at the end of the embedded bar. This equation checks the tensile strength of the wood perpendicular to the plane of the steel threaded rod. An ultimate capacity for wood fracture can be found in Equation 5-4:

$$(\varphi Q_n)_{wood} = \varphi_{conn} k_1 A_w f_t \quad (\varphi Q_n)_{wood} = \varphi_{conn} k_1 A_w f_t \qquad \textbf{Equation 5-4}$$

φ φ = Strength reduction factor (from NZS 3603:1993, Assume 1.0 for ultimate strength)

Q_n Q_n = Nominal axial strength of the connection (kN)

φ_{conn} φ_{conn} = 0.7 (Assume 1.0 for ultimate strength)

k_1 k_1 = Duration load factor (Assume 1.0 for ultimate strength)

A_w A_w = Net area of wood cross section, excluding drilled holes ($mm^2/1000$)

f_t f_t = Characteristic tensile strength (MPa)

Inserting cold testing conditions for epoxy grouted specimens gives the following equation:

$$(1.0Q_n)_{wood} = (1.0)(1.0)\left(63mm \times 150mm - \left(\pi 18mm^2\right)\right)(30MPa)$$

$$(Q_n)_{wood} = 253kN$$

A specimen size of 63mm x 150mm combined with a tensile strength of 30MPa requires a force of 253kN to cause wood fracture. Similar to the case for steel yielding, the high force required suggests that wood fracture is an unlikely failure scenario.

The final check involved bar pull out from the epoxy grouted steel to wood specimen. This failure mechanism coincides with a Mode 3 failure; pull out of the steel threaded rod from the LVL section. The Timber Design Guide provides a two part equation to check for bar pullout, including Equation 5-5 to account for group action of epoxy grouted bars, and Equation 5-6 calculating the pull out strength of a single steel threaded rod embedded in wood. The first step for group action of epoxy bars can be calculated as:

$$(\varphi Q_n)_{pullout} = \varphi_{conn} k_1 n k_g Q_k$$ **Equation 5-5**

φ φ = Strength reduction factor (from NZS 3603:1993, Assume 1.0 for ultimate strength)

Q_n Q_n = Nominal axial strength of the connection (kN)

φ_{conn} φ_{conn} = 0.7 (Assume 1.0 for ultimate strength)

k_1 k_1 = Duration load factor (Assume 1.0 for ultimate strength)

n_k n = Number of steel bars

k_g k_g = Bar group reduction factor (1.0 for 2 bars or less)

Q_k Q_k = Characteristic axial capacity of bar considering pull out (kN)

To calculate the characteristic axial capacity of a single bar considering a pull out failure, Equation 5-6 can be used for analysis:

$$Q_k = 6.73 k_b k_e k_m \left(\frac{l}{d}\right)^{0.86} \left(\frac{d}{20}\right)^{1.62} \left(\frac{h}{d}\right)^{0.5} \left(\frac{e}{d}\right)^{0.5}$$ **Equation 5-6**

Q_k Q_k = Characteristic axial capacity of bar considering pull out (kN)

k_b k_b = Bar type factor (Assume 1.0 for threaded steel rod)

k_e k_e = Epoxy factor (Assume 1.0 for all epoxy types)

k_m k_m = Moisture factor (Assume 1.0 for moisture content less than 15%)

l l = Embedment length (mm) (5d ≤ l ≤ 20d)

d d = Steel bar diameter (mm) (12mm ≤ d ≤ 24mm)

h h = Hole diameter (mm) (1.15d ≤ h ≤ 1.4d)

e e = Edge distance from centre of bar (mm) (e ≥ 1.5d)

Inserting testing parameters into the Timber Design Guide equation gives the following equation checking for bar pull out capacity:

$$Q_k = 6.73(1.0)(1.0)(1.0)\left(\frac{150mm}{16mm}\right)^{0.86}\left(\frac{16mm}{20}\right)^{1.62}\left(\frac{18mm}{16mm}\right)^{0.5}\left(\frac{31.5mm}{16mm}\right)^{0.5}$$

$$Q_k = 47.8kN$$

A pull out load of nearly 48kN can be expected as the ultimate load using the bar pull out failure equation from the Timber Design Guide. Inputting the equation for a single bar into Equation 5-5 gives the following:

$$\left[(1.0)Q_n\right]_{pullout} = (1.0)(1.0)(1.0)(1.0)47.8kN$$

$$(Q_n)_{pullout} = 47.8kN$$

The ultimate failure load for bar pull out with a single bar is nearly 48kN. This assumes that all load and safety factors are equal to 1.0 and that values are equivalent for use in LVL in addition to glulam wood embedment. Further, checking the embedment hole diameter parameter reveals that the test specimen hole diameter is under-sized. A minimum of 1.15d, equal to 18.4mm, is slightly larger than the 18mm embedment hole used in cold testing. This difference, however, is assumed to be negligible for comparison.

A review of the Timber Design Guide bar pull out value compared to experimental values indicates that ultimate strengths for each of the three specimens are considerably greater than the pull out failure load (as seen in Figure 5-15). Differences in type of wood used could contribute to the disparity. Despite efforts to attain the ultimate failure load, assuming strength reduction and safety factors equal to 1.0, it is possible that conservative values still remain within the bar pull out equation. One justification for differences is the selection of the equation coefficient, which is discussed below.

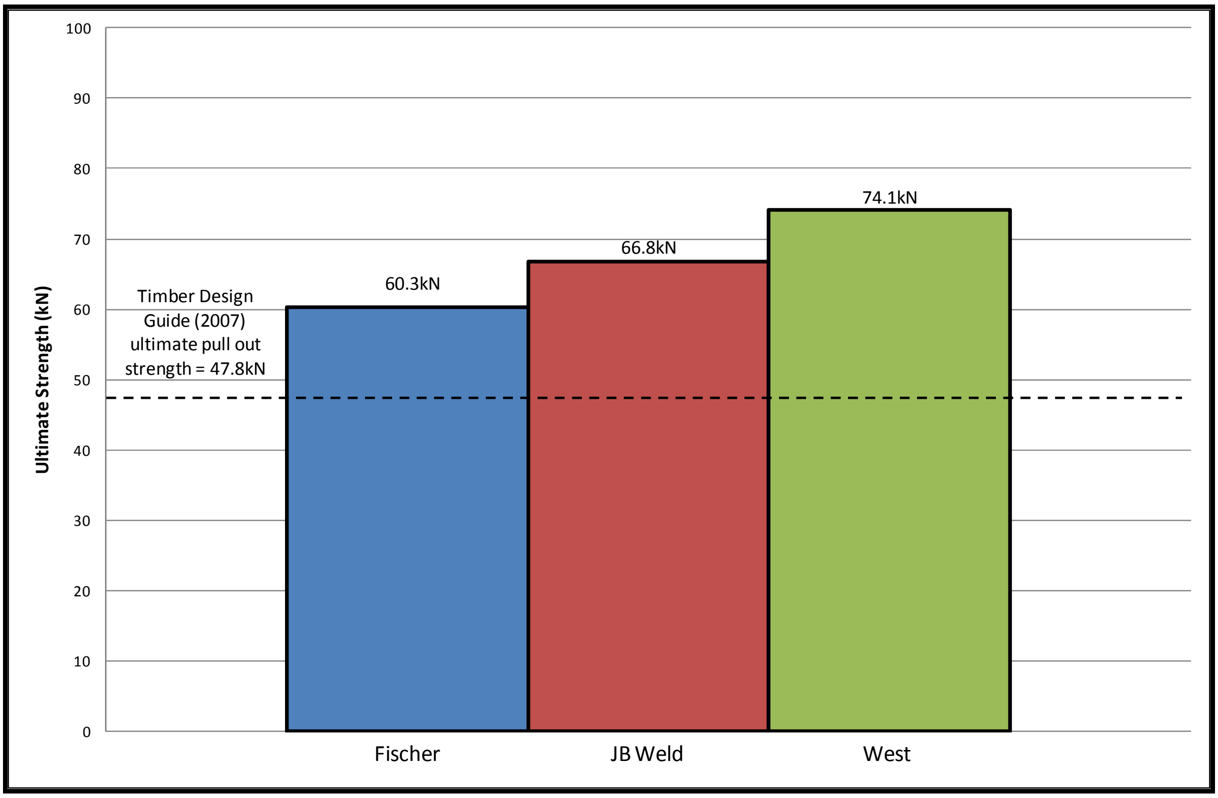

Figure 5-15 – Cold Test - Timber Design Guide Bar Pull Out Failure Comparison

Comparing the Timber Design Guide ultimate load for bar pull out failure and the results from Deng (1997) indicates extreme similarities. Due to their identical nature, it appears as though the Timber Design Guide referenced work from Deng. The only difference between the two values is the equation coefficient, changing from 10.94 in Deng's equation to 6.73 in the Timber Design Guide. Deng (1997) explains that the coefficient was used as part of a unit conversion assessment and to match experimental data for evaluation. The Timber Design Guide, however, uses a 5th percentile characteristic strength value to mandate conservative ultimate strength values for epoxy grouted connections.

5.5.2. Timberlinx Specimens

Product data found on the Timberlinx website (Preston, 2006) provided design tensile load data for comparison with cold test results.

5.5.2.1. Preston (2006)

Previous tensile testing of Timberlinx 'A475' steel to wood connections was performed with 140mm x 140mm sections of White Pine timber. Design values for limit state design were suggested for tensile load parallel to grain (laminations), equal to 12.0kN for standard-term loading and 13.7kN for short-term loading. Assuming values for standard-term loading, a design tensile load of 12.0kN has been identified for comparison with the 36.0kN ultimate load in cold testing in Figure 5-16:

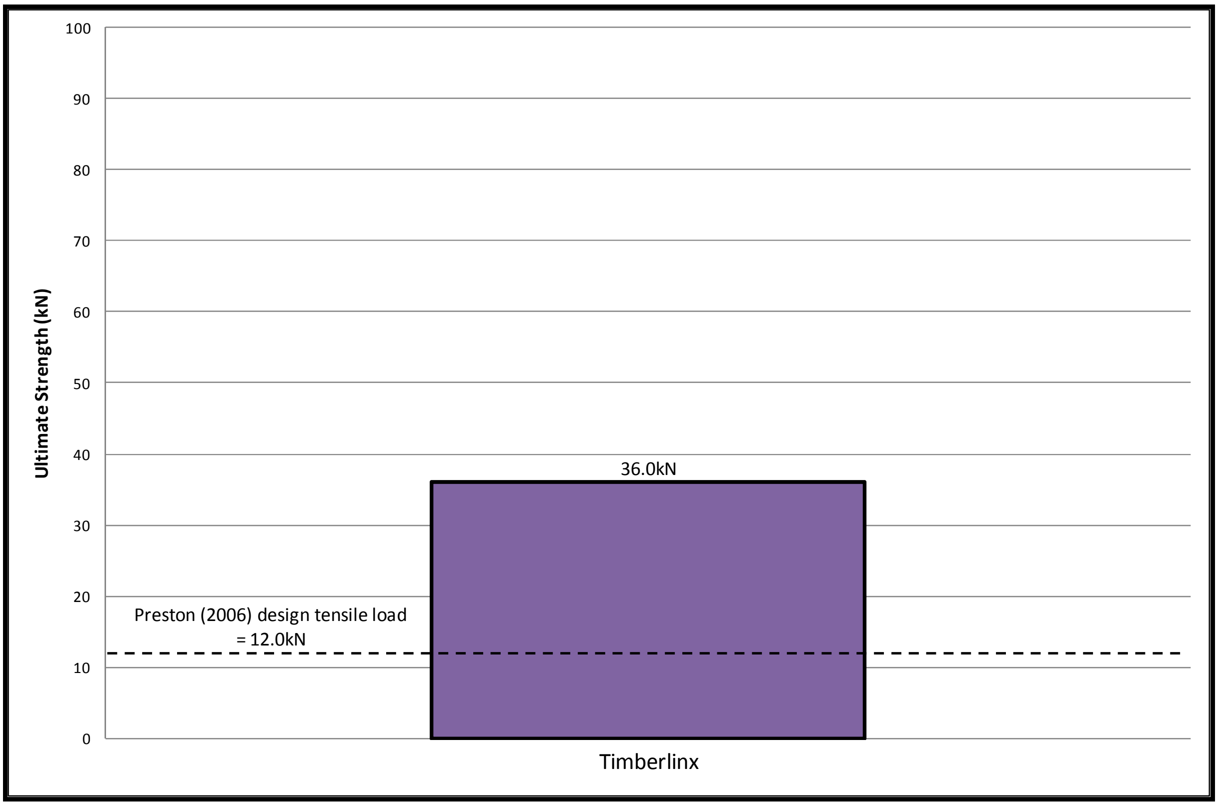

Figure 5-16 – Cold Test - Preston (2006) Comparison

Several possible factors may explain the difference between the experimentally derived ultimate load compared to the prescribed design tensile load. The primary factor would be due to strength reductions and safety factors inherently included in design values. The design load could have been significantly reduced compared to the ultimate load by including reduction factors for the design strength. In addition, the use of LVL as opposed to White Pine timber used in testing would increase the strength of the connection, as the failure mechanism is based on the strength of the wood.

5.5.3. Lagscrewbolt Specimens

Results from pull out tests performed by Nakatani (2009) provided data for comparison with experimental cold testing results. While specimen sections varied slightly, ultimate behaviour proved to be identical, with similar failure mechanisms despite differing ultimate loads.

5.5.3.1. Nakatani (2009)

Tensile pull out tests performed by Nakatani (2009) at the University of Auckland established an average ultimate tensile load for Lagscrewbolt sections embedded in LVL. 200mm Lagscrewbolt products (identical to the size used in cold testing) were embedded in 100mm x 100mm Radiata Pine LVL parallel to the laminations (identical to orientation used in cold testing). Specimens were subjected to increasing tensile load until failure. Results for the average of four tests compared to cold tests results are shown in Figure 5-17:

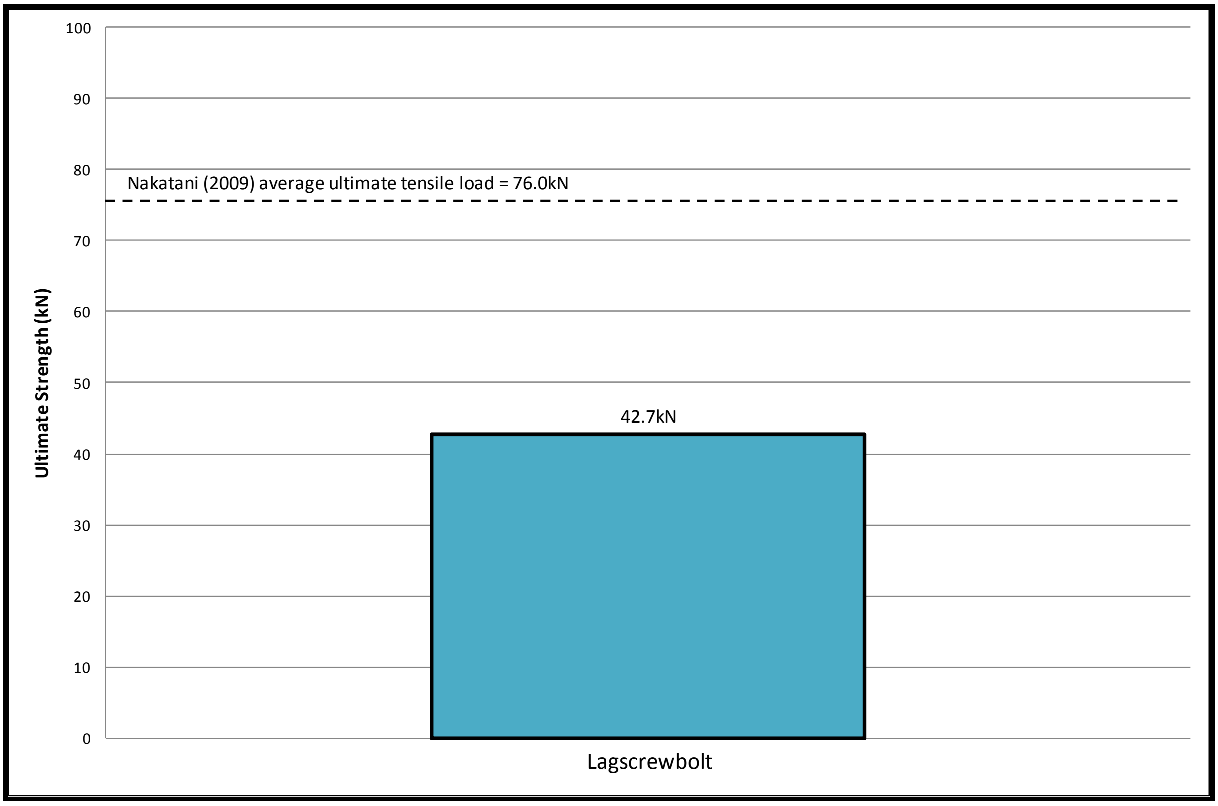

Figure 5-17 – Cold Test - Nakatani (2009) Comparison

The average tensile load submitted by Nakatani (2009) is significantly higher than the ultimate load found through cold testing. This could be due to several reasons, the first of which attributed to the LVL section size. Similar to the Timberlinx specimens, it was expected that wood would be the critical failure mechanism. Having a larger section of wood (100mm x 100mm compared to 63mm x 150mm) would dictate greater ultimate tensile strength.

Nakatani (2009) indicates that two of the six specimens tested experienced failure due to cracking at the glue-line. The other four specimens failed due to Lagscrewbolt pull out. Nakatani explains that test results for specimens demonstrating the glue-line failure were not included in the average failure load. This resulted in higher average ultimate tensile strength for experimental specimens.

Observing that a glue-line failure is similar to a Mode 1 Failure suggests that this failure mechanism may be more common than previously anticipated. A glue-line failure could be an indicator of reduced ultimate tensile load compared to the alternative Lagscrewbolt pull out failure. This difference in failure mechanism could be a contributing factor for justifying the reduced load for cold testing compared to previous testing.

5.6. Conclusions

Cold testing data provided benchmark ultimate loads and failure methods for epoxy grouted steel to wood specimens and proprietary mechanical fasteners. The data can be used for comparison with previous studies to evaluate differences in epoxy types.

5.6.1. Epoxy Specimens

Cold testing data for epoxy grouted steel threaded rod specimens in LVL ranged from 58kN to 75kN. Experimental ultimate loads compared favourably with analysis techniques for predicting the ultimate failure load, despite the fact that some equations were intended for use in different types of wood.

An analysis of failure modes revealed two distinct failure mechanisms for the epoxy grouted steel to wood specimen. Cold testing observed a confinement failure with splitting perpendicular to the veneers as well as bar pull out failure within test specimens.

5.6.2. Timberlinx Specimens

A comparison of cold testing data with published literature for the Timberlinx steel to wood connector revealed a surprising strength difference of almost three times greater for cold testing. This can be attributed to significant reductions resulting from strength and safety factors. For design purposes, it is prudent to ensure a level of conservatism to maintain safety for Timberlinx connector use.

A lack of failure method data from published literature makes a comparison difficult. Despite this lack of data, it was anticipated that a wood failure would be the most likely failure mechanism. The strength of the steel connector greatly surpasses the strength of the LVL, forcing a wood failure in the Timberlinx test specimen.

5.6.3. Lagscrewbolt Specimens

Experimental cold testing with Lagscrewbolt provided surprisingly low ultimate strength values compared to published testing data. The ultimate load for cold tests was nearly half of the average ultimate load suggested in previous pull out tests of Lagscrewbolt samples in LVL. This is possibly due to the differences in wood product used, LVL specimen size, and failure mechanism.

Differences in failure mechanism could contribute to the ultimate load discrepancy. Testing performed by Nakatani demonstrated confinement failures, similar to cold testing. Tensile strength values from these test specimens resulted in reduced strength. Nakatani did not include these ultimate loads as part of the average ultimate tensile strength for Lagscrewbolt connections. This resulted in a greater average ultimate tensile strength for previously tested specimens. Additional testing may evaluate the effects of confinement failure compared to a potential pull out failure for Lagscrewbolt ultimate strength.

6. Oven Testing

The second phase of experimentation, testing steel to wood connections at high temperatures, was used to determine the ultimate strength of epoxy and proprietary connections at temperatures ranging from 50°C to 200°C. Both ultimate strengths and failure modes were compared with results from cold testing to determine the heating effect on steel to wood connections.

6.1. Oven Test Specimens

Oven test specimens were constructed exactly the same as cold test specimens. Each specimen was composed of NelsonPine LVL with M16 Grade 8.8 high strength steel threaded rod. The Timberlinx and Lagscrewbolt proprietary mechanical fasteners used steel components for connection strength. All specimens were heated in the University of Canterbury Civil Engineering oven and tested in the Avery Testing Machine using the same testing procedure.

6.1.1. Epoxy Specimens

Epoxy specimens used for oven testing were identical to specimens used in cold testing. Additional information on epoxy specimens used in oven tests can be found in Section 5.1.1. Diagrams of epoxy specimens used in oven testing can be found in Figure 5-1 and Figure 5-2 in the Cold Testing section.

6.1.2. Timberlinx Specimens

Timberlinx specimens for oven testing were constructed to the same specifications as cold testing specimens. Further information on Timberlinx oven test specimens including diagrams of the Timberlinx 'A475' steel to wood connector can be found Section 5.1.2 and Figure 5-3 and Figure 5-4, respectively.

6.1.3. Lagscrewbolt Specimens

Testing specimens using the Lagscrewbolt steel to wood connector for oven tests were built using the same procedure as the epoxy and Timberlinx specimens. Additional information on Lagscrewbolt oven test specimens can be found in Section 5.1.3. Schematic diagrams of the Lagscrewbolt product can be seen in Figure 5-5 and Figure 5-6.

6.2. Testing Procedure

Oven testing was performed in the University of Canterbury Civil Engineering Laboratory with the Avery Testing Machine to determine the ultimate tensile strength of steel to wood connections at high temperatures. The oven testing procedure involved the same testing procedure as with cold testing, however test specimens were heated in the oven prior to testing.

6.2.1. Oven Heating

Steel to wood connector samples were inserted into the Structures Extension Laboratory oven (shown in Figure 6-1) overnight and heated to temperatures ranging from 50°C to 200°C at 50°C intervals for testing. Previous testing determined that heating specimens overnight provided sufficient time for consistent heating throughout the entire specimen (Harris, 2004).

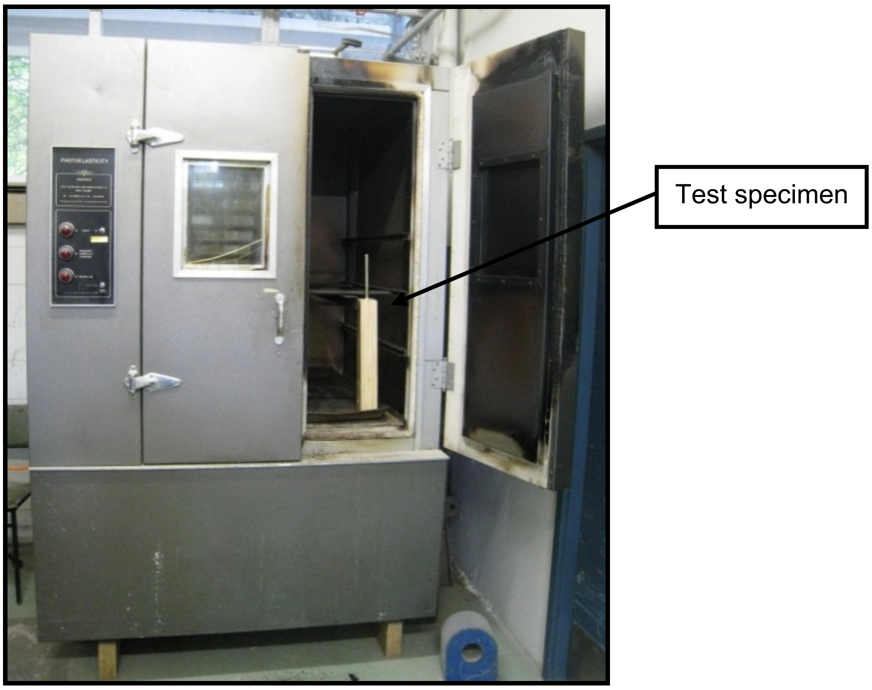

Figure 6-1 – Oven test Specimen Heating

6.2.2. Specimen Preparation

Specimen preparation for oven samples proceeded similar to the process previously outlined in Section 5.2.1. Heated test specimens were wrapped in a leather blanket (shown in Figure 6-2) following removal from the oven to minimize heat loss prior to testing.

Figure 6-2 – Wrapped Oven Test Specimen

Wrapped testing specimens were equipped with the custom steel bracket at the free end to provide support in the Avery Testing Machine. The leather blanket was positioned to provide maximum insulation from heat loss (shown in Figure 6-3) while the custom steel bracket was being attached.

Figure 6-3 – Oven Test Specimen Preparation

6.2.3. Load Application

Tensile testing of oven specimens was performed identical to cold tests. Further information regarding load application can be found in Section 5.2.2.

6.2.4. Data Recording

Experimental data was recorded by the Universal Data Logger (UDL) software programmed on the University of Canterbury Civil Engineering Laboratory computer. The ultimate tensile load was digitally recorded throughout the duration of each tensile test from initial loading to failure. Additional information can be found regarding data recording in Section 5.2.3.

6.2.5. Specimen Failure

Specimen failure for oven tests was defined the same as in cold tests: limited to the strength domain evaluating the ultimate tensile load. Failure was defined as the point at which the connection was unable to sustain additional tensile load. A further discussion of specimen failure can be found in Section 5.2.4.

6.3. Results

Oven testing in the University of Canterbury Civil Engineering Laboratory was performed to enhance the understanding of heating effects on steel to wood connections. Heated specimens were subjected to tensile testing to determine the ultimate failure load at high temperatures. Results were used to evaluate the ultimate strength of steel to wood connections tested at elevated temperatures.

Results for ultimate load values are shown in Table 6-1, Figure 6-4 and Figure 6-5. A summary of failure modes is shown in Table 6-2:

Temperature	Fischer	JB Weld	West	Timberlinx	Lagscrewbolt
50°C	46.2	41.2	44.9	39.0	35.1
100°C	34.6	25.9	13.2	24.6	33.6
150°C	27.9	23.1	9.4	25.7	29.0
200°C	22.8	21.1	12.5	15.8	32.4

Table 6-1 – Oven Test Ultimate Strength Results (kN)

Temperature	Fischer	JB Weld	West	Timberlinx	Lagscrewbolt
50°C	1	1	1	2	1
100°C	4	4	4	2	1
150°C	4	4	4	2	5
200°C	4	4	4	2	5

Table 6-2 – Oven Test Failure Mode Results

Oven test results indicate epoxy grouted steel to wood connections provide greater strength at high temperatures compared to proprietary mechanical fasteners. Each type of connector, however, exhibits differing behaviour with increase in temperature. High temperature epoxy specimens display a considerable decrease in ultimate load with increasing temperature. Proprietary mechanical fasteners demonstrate gradual strength decreases at higher temperatures. Furthermore, the failure modes for epoxy test specimens change with increasing temperature, while proprietary mechanical fasteners remain mostly constant.

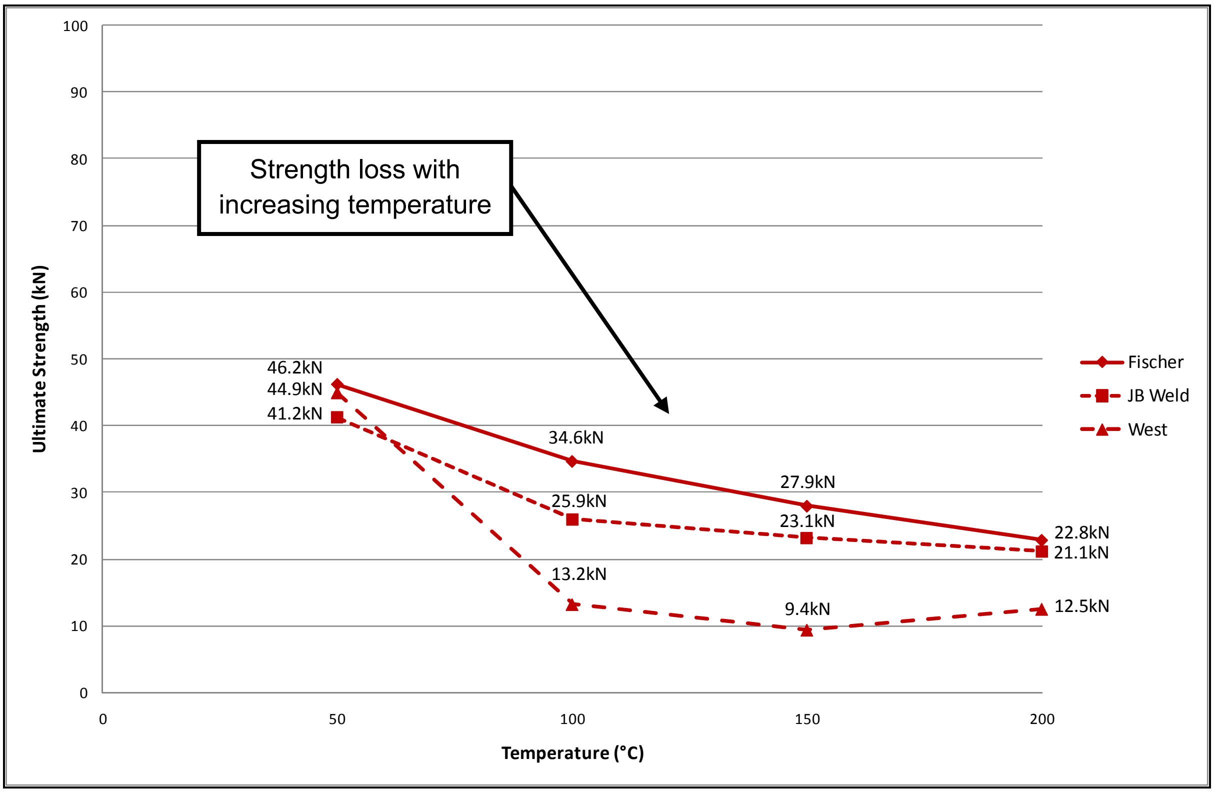

Figure 6-4 – Oven Test Epoxy Specimen Results

Epoxy test specimens displayed a decrease in ultimate strength with an increase in temperature. Despite being labelled as high temperature epoxies, each of the epoxy test specimens experienced a loss of strength with temperature. The West epoxy displayed the most significant loss of strength, with a 70% reduction in ultimate load from 50°C to 100°C. The Fischer and JB Weld epoxies, however, demonstrated a more gradual strength reduction as opposed to immediate loss of strength at the previously established 100°C plateau (Barber, 1994).

High temperature epoxy test specimens exhibited two unique failure mechanisms across the experimental temperature protocol (as seen in Table 6-2). While all failure methods remained brittle failures, test specimens at 50°C demonstrated wood failures, similar to cold testing. Beyond 50°C, however, test specimens displayed epoxy failures. The transition in failure mechanism was likely the result of physical property changes in the epoxy at high temperatures.

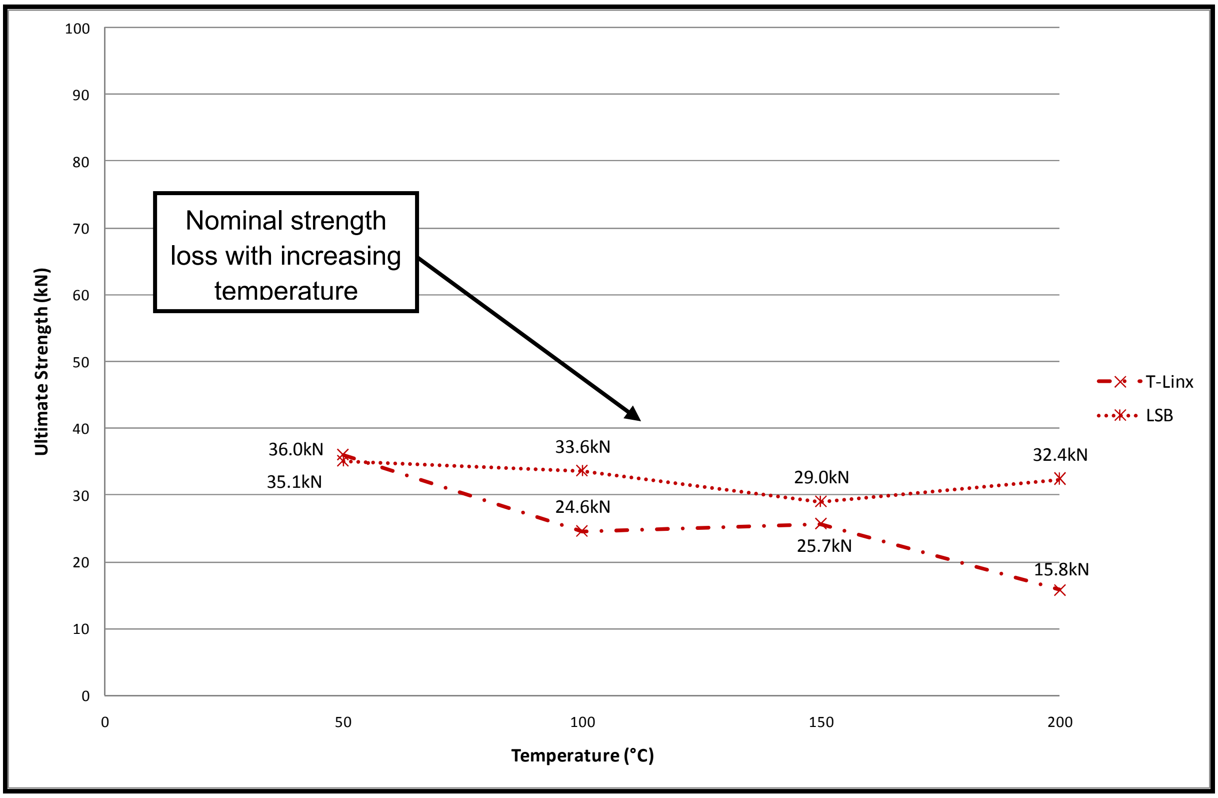

Figure 6-5 – Oven Test Proprietary Specimen Results

Results for the Timberlinx and Lagscrewbolt proprietary test specimens followed the trend of decreasing ultimate strength with increasing temperature. This was likely due to a loss of strength in the LVL as opposed to loss of strength in the connection. The failure mechanism for Timberlinx test specimens remained constant regardless of temperature. Lagscrewbolt specimens displayed differing failure modes between 100°C and 150°C, transitioning from a confinement failure to a pull out failure.

6.4. Failure Modes

Oven testing specimens revealed four failure modes occurring during tensile testing at high temperatures. The first two failure modes were witnessed in cold testing, with additional failure modes, Mode 4 and Mode 5 Failures, observed in epoxy grouted steel to wood specimens at high temperatures. Each of the four failure modes presented brittle failure mechanisms occurring within the structural wood or epoxy.

6.4.1. Mode 1 Failure

A Mode 1 Failure, evident in cold testing specimens, represents a brittle confinement failure in the LVL specimen perpendicular to the laminations. Mode 1 Failures in oven test specimens (shown in Figure 6-6) were evident at lower temperatures, occurring with the Fischer and JB Weld epoxies tested at 50°C. Failure of the wood was not apparent at higher temperatures, as the epoxy strength became the primary failure mechanism at high temperatures. Additional information on Mode 1 Failures can be found in Section 5.4.1.

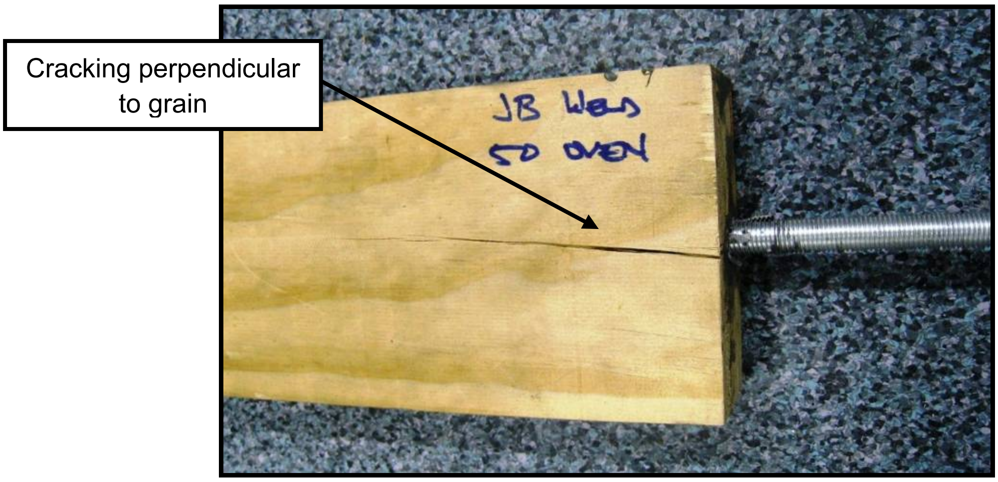

Figure 6-6 – Mode 1 Failure in JB Weld 50°C Oven Specimen

6.4.2. Mode 2 Failure

A Mode 2 Failure (seen in Figure 6-7), as observed in cold tests with the Timberlinx specimen, occurred when the tensile force caused a brittle failure of the LVL specimen parallel to the laminations in line with the expanding anchor. Comparison with previous resources was not possible, as tensile testing of the Timberlinx connector at high temperatures is unique to this research. Further information regarding Mode 2 Failures can be found in Section 5.4.2.

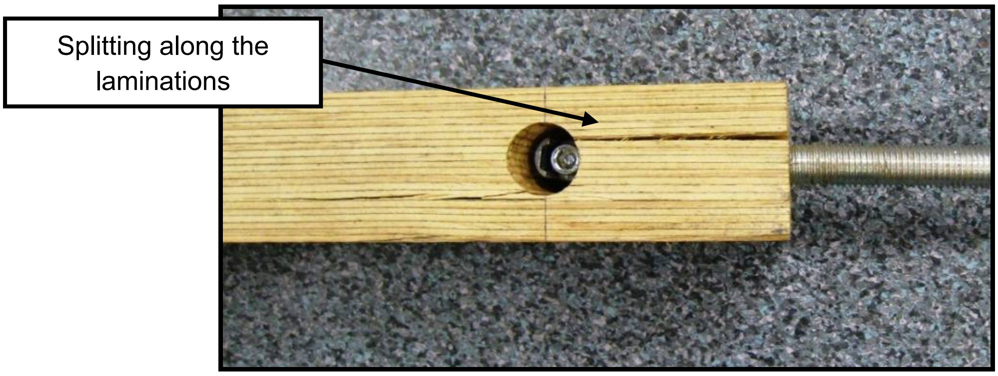

Figure 6-7 – Mode 2 Failure in Timberlinx 50°C Oven Specimen

6.4.3. Mode 4 Failure

A Mode 4 Failure was the most common failure method observed in oven testing. As the tensile load on the steel to wood connection increased, the steel threaded rod and grouted epoxy adhesive pulled out at the epoxy-wood interface resulting in a sudden, brittle failure. Heating the test specimens to high temperatures resulted in thermal degradation of the epoxy adhesive, causing it to turn to a powder and crumble when heated and subjected to tensile load.

Mode 4 Failures were present in all epoxy test specimens subjected to oven heating of 100°C and above. Test results indicate that epoxy test specimens undergo a physical transformation in properties when heated to high temperatures, evidenced by the transition from Mode 1 Failures at 50°C to Mode 4 Failures at 100°C and above.

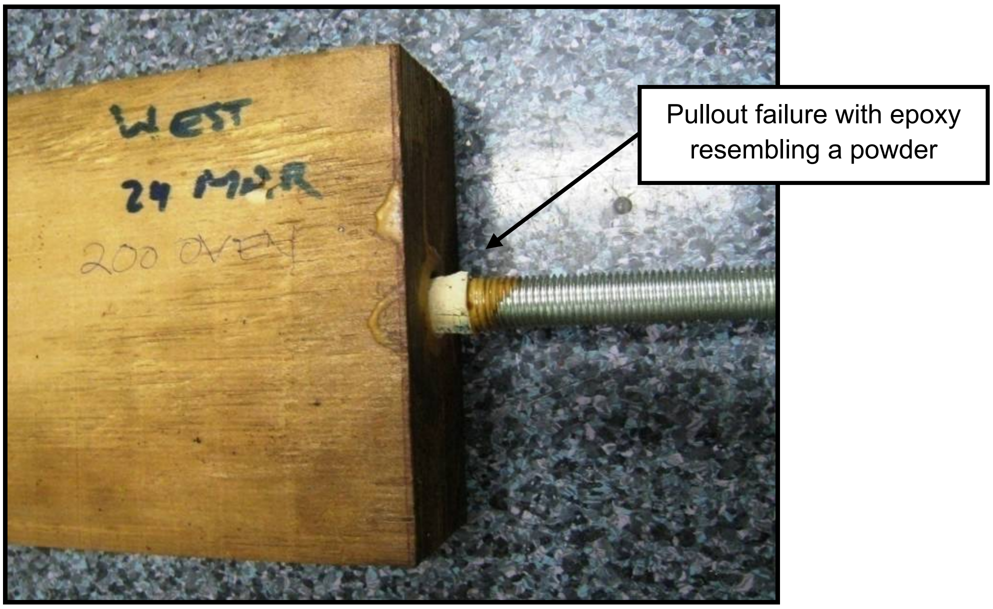

Figure 6-8 – Mode 4 Failure in West 200°C Oven Specimen

Failed specimens displayed an epoxy adhesive cylinder (as shown in Figure 6-8) around the steel threaded rod, located at the epoxy-wood interface. The epoxy appeared powdery and displayed no removal of wood on the withdrawn epoxy grouted steel rod. Thermal effects reduced the epoxy adhesive strength at a greater rate than the tensile strength of the LVL, causing it to become the dominant failure mechanism. This suggests that the wood maintained sufficient strength at high temperatures, forcing failure to occur within the epoxy at the epoxy-wood interface.

Failure occurring at the epoxy-wood interface with the epoxy turning powdery and crumbling is consistent with findings from oven tests performed by Harris (2004). Harris witnessed identical thermal degradation of epoxy adhesive in all-purpose epoxy grouted steel rod tests at high temperatures ranging from 50°C to 100°C.

6.4.4. Mode 5 Failure

The predominant failure method for Lagscrewbolt samples at high temperatures was a Mode 5 failure. The Lagscrewbolt specimen pulled out from the LVL section, removing a significant amount of wood between the steel threads (as shown in Figure 6-9). This brittle failure occurred as the tensile load on the specimen increased, causing a shear stress failure at the wood sections between the steel threads. The Mode 5 failure, observed in 50°C and 100°C oven tests samples, can be characterized as a wood pull out failure occurring at the steel threaded rod-wood interface.

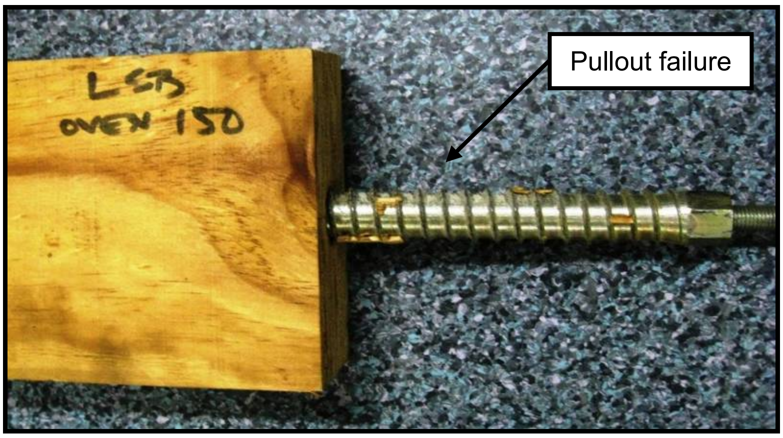

Figure 6-9 – Mode 5 Failure in Lagscrewbolt 150°C Oven Specimen

Given the behaviour of the Lagscrewbolt specimens at high temperatures, a comparison with previous resources would be a valuable exercise. However, experimentation with Lagscrewbolt specimens at high temperatures is unique to this research, suggesting the need for additional testing.

6.5. Comparison with Cold Testing

Experimental results from tensile testing of specimens at high temperatures were compared to cold test results to evaluate the performance and behaviour of steel to wood connections. Comparing both the ultimate strength values and failure modes offers insight into the severity of strength loss with increasing temperature.

6.5.1. Epoxy Specimens

All three epoxy grouted steel to wood connections were used in oven testing. These included the Fischer 'FIS V 360 S' Injection Mortar, the JB Weld 'Industro Weld', and the West Systems 'Z206' Epoxy Hardener. Results from oven testing were compared with cold testing to establish high temperature epoxy behaviour at elevated temperatures.

6.5.1.1. Fischer 'FIS V 360 S' Injection Mortar

The Fischer high temperature epoxy exhibited a decrease in ultimate strength with an increase in temperature. Ultimate load values obtained through oven testing are presented with cold testing data in Table 6-3 and Figure 6-10:

Temperature	Ultimate Load (kN)	Ultimate Load / Ambient Ultimate Load (%)
20°C	60.3	100
50°C	46.2	76.5
100°C	34.6	57.4
150°C	27.9	46.3
200°C	22.8	37.8

Table 6-3 – Fischer Cold and Oven Test Results

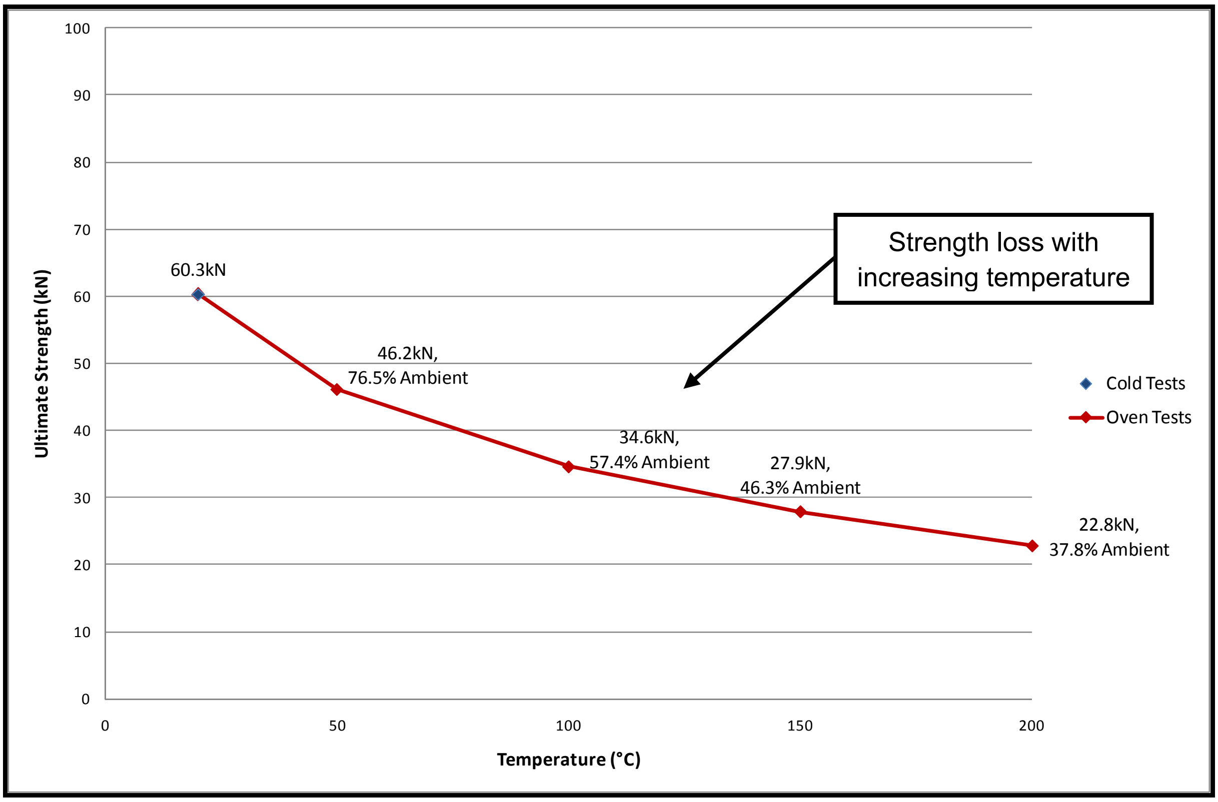

Figure 6-10 – Fischer Cold and Oven Test Results

Table 6-3 displays the ultimate load values for cold and oven testing. The table also displays the strength percentage maintained for oven test samples compared to the ambient temperature control sample. While previous experimentation suggested epoxy specimens are unable to maintain strength beyond 100°C, results indicate that the Fischer high temperature epoxy was able to maintain nearly 60% of the ultimate design strength at this 100°C plateau. Furthermore, the Fischer epoxy sample maintained nearly 40% of the ambient ultimate design strength at 200°C.

Observing the ultimate load values for cold and oven test specimens in a graphical display demonstrates the rate of strength loss with increasing temperature. Analyzing the data presented in Figure 6-10 suggests that the Fischer high temperature epoxy displays a gradual strength loss with temperature. This is evidenced by the relatively steady slope connecting the data points on the graph. From ambient conditions at 20°C to maximum heated conditions at 200°C, the Fischer epoxy maintains a consistent strength loss across the temperature range. Additional testing could quantify ultimate strength values within this range.

Observing the failure modes reveals changes in failure mechanism and specimen behaviour at high temperatures. Oven tests were engaged to determine both the behaviour and failure mechanism of steel to wood connections at high temperatures. Failure modes for Fischer cold and oven test specimens are shown in Table 6-4:

Temperature	Failure Mode
20°C	1
50°C	1
100°C	4
150°C	4
200°C	4

Table 6-4 – Fischer Cold and Oven Test Failure Modes

Specimens tested at ambient temperature and heated to 50°C both displayed Failure Mode 1, a confinement failure of the LVL attributed to cracking perpendicular to grain. Tests specimens heated beyond 50°C displayed a different failure mode: Failure Mode 4, an epoxy failure at the epoxy-wood interface. Failure Mode 4 was readily identified by pull out of the epoxy grouted steel threaded rod and the high temperature epoxy turning powdery in consistency. Further information on failure modes can be found in Section 5.4.1 for Failure Mode 1 and Section 6.4.3 for Failure Mode 4.

The difference in failure mechanism suggests that heating the Fischer test specimens beyond 50°C causes a strength loss in the epoxy. The epoxy changes from the solid mass seen in cold tests to a powdery substance that can be easily brushed off after failure. As the temperature increases, an LVL confinement failure is no longer the critical mechanism. Pullout of the heated, strength-reduced, high temperature epoxy becomes the dominant failure mechanism at high temperatures.

6.5.1.2. JB Weld 'Industro Weld'

The JB Weld 'Industro Weld' displayed strength loss with increase in temperature. Ultimate load values obtained through tensile testing at ambient and elevated temperatures are shown in Table 6-5 and Figure 6-11:

Temperature	Ultimate Load (kN)	Ultimate Load / Ambient Ultimate Load (%)
20°C	66.8	100
50°C	41.2	61.7
100°C	25.9	38.7
150°C	23.1	34.6
200°C	21.1	31.6

Table 6-5 – JB Weld Cold and Oven Test Results

Ultimate loads, displayed in Table 6-5, demonstrate that the JB Weld epoxy experienced a significant strength decrease in the initial test at 50°C, maintaining about 60% of ultimate strength compared to testing at ambient temperature. When heated to 100°C, JB Weld specimens maintained nearly 40% of ambient strength, decreasing to just over 30% of ambient strength maintained at 200°C.

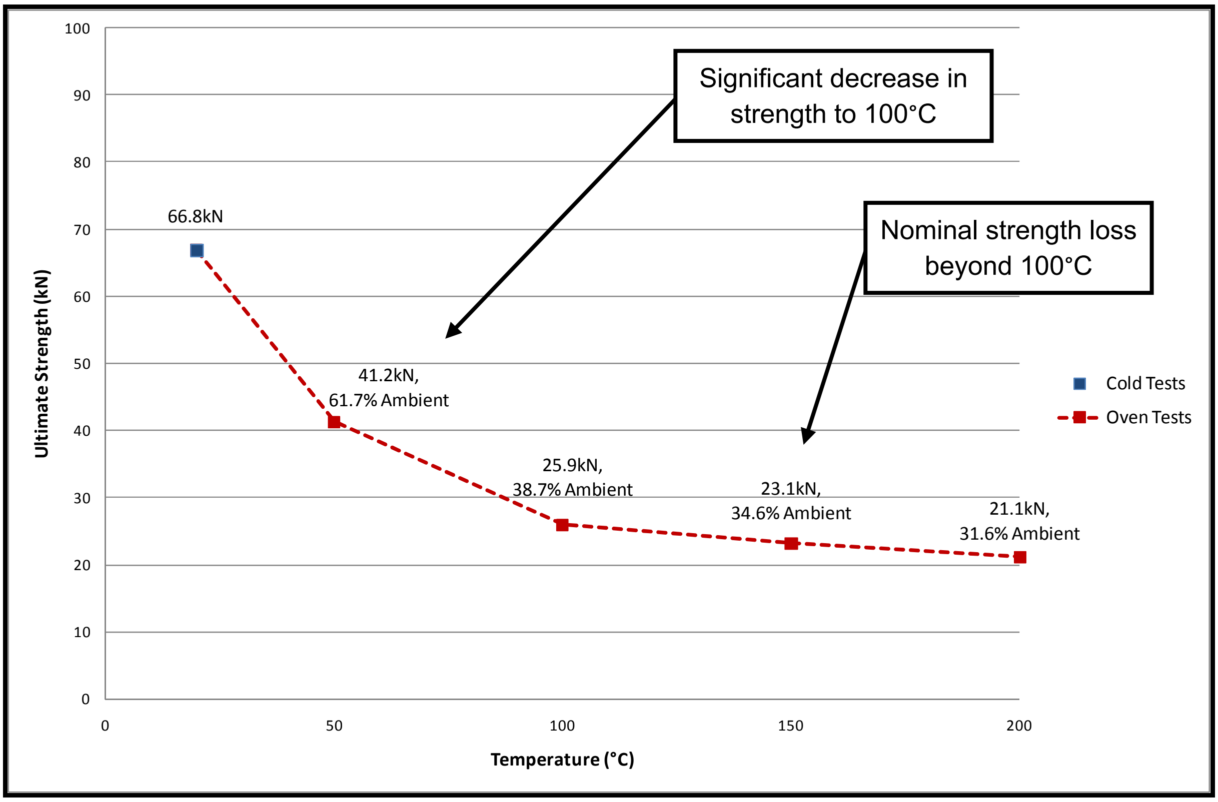

Figure 6-11 – JB Weld Cold and Oven Test Results

The slope in Figure 6-11 demonstrates the rate of change of strength loss with increasing temperature. The JB Weld epoxy appears to lose a significant amount of strength when tested to the previously established limit of 100°C (Barber, 1994). However, the epoxy demonstrates unexpected behaviour by maintaining strength beyond the 100°C temperature plateau. This gradual loss of strength at high temperatures suggests that the JB Weld epoxy can be expected to initially lose strength when heated to high temperatures, yet strength loss curtails to a limit. Further testing may provide additional guidance for an expected amount of strength maintained at high temperatures.

Comparing the failure modes for JB Weld specimens tested at high temperatures reveals changes in the dominant failure mechanism. Failure modes for the JB Weld epoxy are presented in Table 6-6:

Temperature	Failure Mode
20°C	3
50°C	1
100°C	4
150°C	4
200°C	4

Table 6-6 – JB Weld Cold and Oven Test Failure Modes

Failure modes for the JB Weld epoxy grouted steel threaded rod connection vary at lower temperatures and become more consistent at high temperatures. The ambient specimen exhibited a Mode 3 Failure: pull out of the steel threaded rod and epoxy with wood attached while the 50°C specimen displayed a Mode 1 Failure: a confinement failure with cracking perpendicular to grain in the LVL.

JB Weld epoxy demonstrates a transition from wood failures to epoxy failures when heated to temperatures exceeding 50°C. Mode 4 Failures, witnessed in 100°C to 200°C test specimens, represent brittle epoxy adhesive failures, where the epoxy turns into a powdery substance and pulls out from the LVL at the epoxy-wood interface.

6.5.1.3. West System 'Z206' Epoxy Hardener and Adhesive Technologies 'ADR 310' Epoxy Resin

Test specimens using West epoxy adhesive displayed a loss of ultimate strength with increase of temperature. Ultimate load values for tensile testing and a comparison between the ambient ultimate load and oven tested ultimate load is shown in Table 6-7:

Temperature	Ultimate Load (kN)	Ultimate Load / Ambient Ultimate Load (%)
20°C	74.3	100
50°C	44.9	60.4
100°C	13.2	17.8
150°C	9.4	12.6
200°C	12.5	16.8

Table 6-7 – West Cold and Oven Test Results

Tensile test results for the West epoxy suggest that a considerable amount of strength was lost prior to reaching the 100°C plateau (Barber, 1994). While the West epoxy demonstrated the greatest ultimate strength value at ambient temperature, only 60% of ambient strength was maintained at 50°C. Less than 20% of strength was maintained at 100°C. Surprisingly, as the temperature increased beyond 100°C, test results do not demonstrate further strength loss. This suggests a minimal amount of strength is preserved in the connection despite the higher temperature.

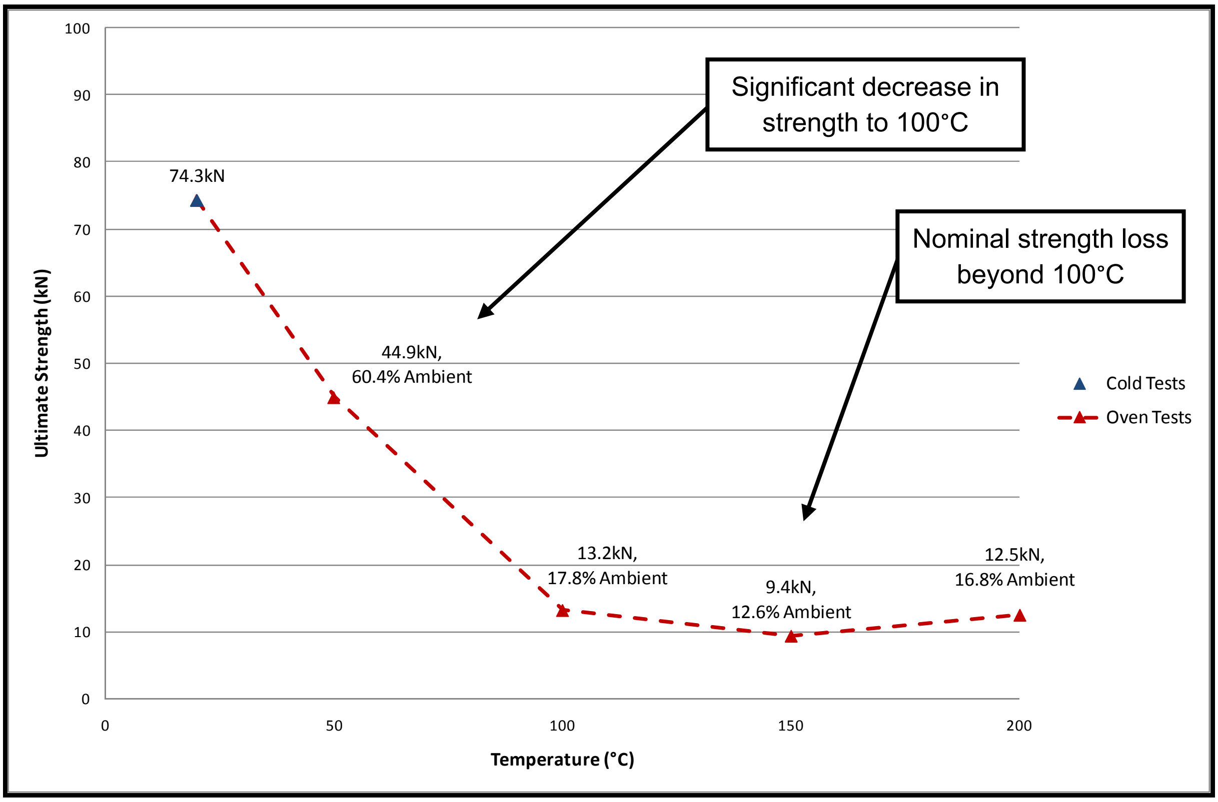

Figure 6-12 – West Cold and Oven Test Results

The slope in Figure 6-12 demonstrates the significant strength loss in West specimens tested between ambient temperature and 100°C. However, the slope appears to stabilise beyond 100°C and remains relatively constant to 200°C. Further testing could validate the theory that West epoxy sustains a minimal amount of strength beyond the 100°C temperature limit.

Comparable to the JB Weld epoxy specimens, West test specimens exhibited the same failure method trend from ambient temperature to 200°C. Failure modes ranged from failures in the LVL at lower temperatures to failures in the epoxy at higher temperatures. A summary of failure modes for West specimens is shown in Table 6-8:

Temperature	Failure Mode
20°C	3
50°C	1
100°C	4
150°C	4
200°C	4

Table 6-8 – West Cold and Oven Test Failure Modes

Wood failures included the Mode 3 Failure: LVL tension failure with steel threaded rod pull out including epoxy and a significant amount of wood, and the Mode 1 Failure: an LVL confinement failure with cracking perpendicular to grain. Each of these failure mechanisms occurred at lower temperatures, suggesting that the load capacity of the wood was less than the capacity of the epoxy.

Higher temperatures, 100°C to 200°C, displayed Mode 4 Failures, epoxy failure occurring at the epoxy-wood interface. At high temperatures, the strength of the epoxy connection decreased and became the dominant failure method.

6.5.2. Timberlinx Specimens

Results from oven testing of proprietary specimens were compared with cold test results to evaluate specimen behaviour and strength at elevated temperatures. Results for ultimate loads of Timberlinx oven test specimens and a comparison with ambient strength is shown in Table 6-9:

Temperature	Ultimate Load (kN)	Ultimate Load / Ambient Ultimate Load (%)
20°C	36.0	100
50°C	39.0	108.3
100°C	24.6	68.3
150°C	25.7	71.4
200°C	15.8	43.9

Table 6-9 – Timberlinx Cold and Oven Test Results

Previous testing of Timberlinx test specimens was limited to tensile testing at ambient temperature (Preston, 2006). Tensile testing at high temperatures provided insight into the magnitude of strength loss associated with increased temperature. Strength loss for Timberlinx specimens embedded in NelsonPine LVL became apparent starting at 100°C, with approximately 70% of the ambient ultimate load maintained. Strength loss remained consistent at 150°C and fell to less than 50% of strength maintained at the maximum tested temperature of 200°C.

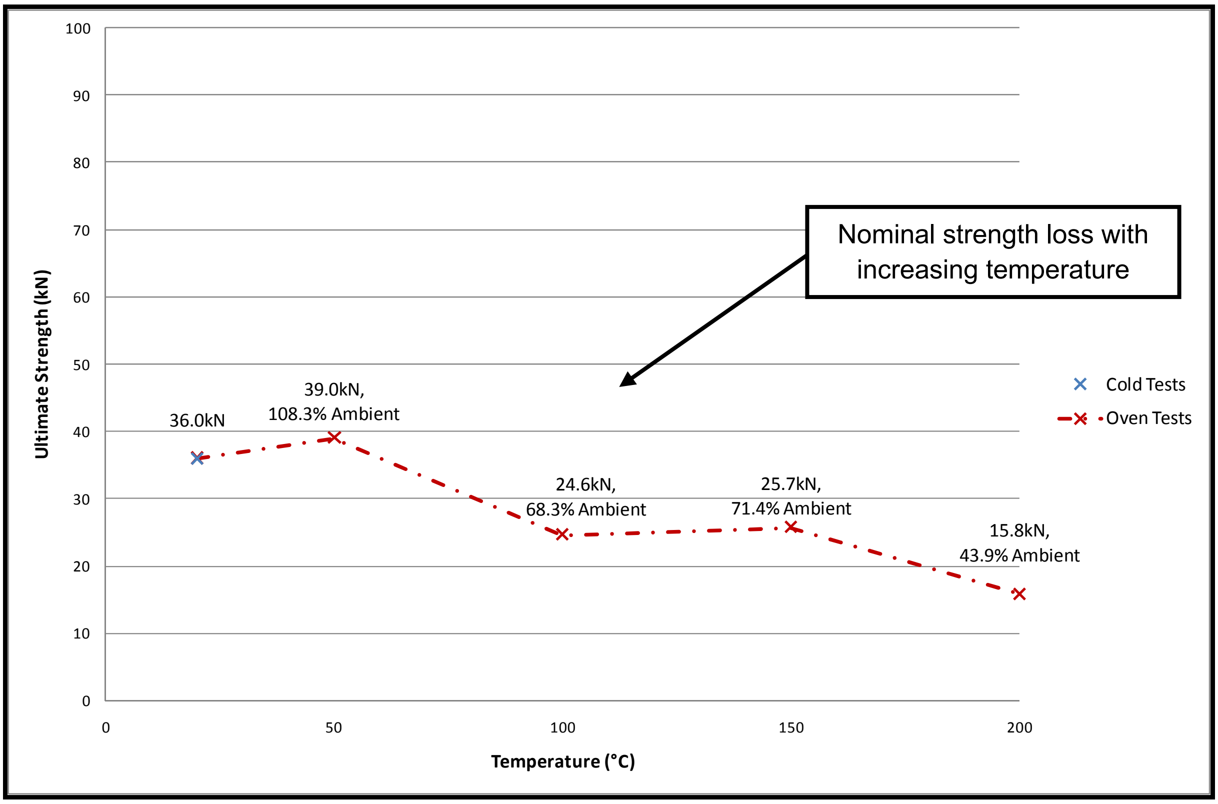

Figure 6-13 – Timberlinx Cold and Oven Test Results

Examining the general slope of ambient and oven test results for the Timberlinx steel to wood connection, as shown in Figure 6-13, reveals the basic trend of strength loss with temperature. Discrepancies between data points could be the result of small sample size, mitigated by additional testing. Further testing could validate the magnitude of strength loss with temperature.

The Timberlinx product presents a unique connection type, resulting in a consistent failure mechanism that appears to be independent of temperature. All test specimens employing the Timberlinx steel to wood connector displayed a Mode 2 Failure; splitting of the LVL along the laminations in line with the expanding anchor bolt. More information on Failure Mode 2 can be found in Section 5.4.2. Results for the failure mode of Timberlinx products tested at ambient and elevated temperatures is shown in Table 6-10:

Temperature	Failure Mode
20°C	2
50°C	2
100°C	2
150°C	2
200°C	2

Table 6-10 – Timberlinx Cold and Oven Test Failure Modes

6.5.3. Lagscrewbolt Specimens

Previous testing of Lagscrewbolt specimens has been limited to tensile testing at ambient temperature (Nakatani, 2009). Results from oven testing are the solitary source of information regarding Lagscrewbolt behaviour at high temperatures. Consistent with oven testing of steel to wood connections, ultimate strength values decreased with increasing temperature. Ultimate strength values are shown in Table 6-11:

Temperature	Ultimate Load (kN)	Ultimate Load / Ambient Ultimate Load (%)
20°C	42.7	100
50°C	35.1	82.3
100°C	33.6	78.8
150°C	29.0	68.0
200°C	32.4	76.0

Table 6-11 – Lagscrewbolt Cold and Oven Test Results

Results from oven testing demonstrate the magnitude of strength loss, with approximately 80% strength maintained at 100°C and 75% strength maintained at 200°C. Compared to alternative steel to wood connection techniques, the Lagscrewbolt displays the greatest ability to maintain ultimate strength at high temperatures when compared to strength at ambient temperature.

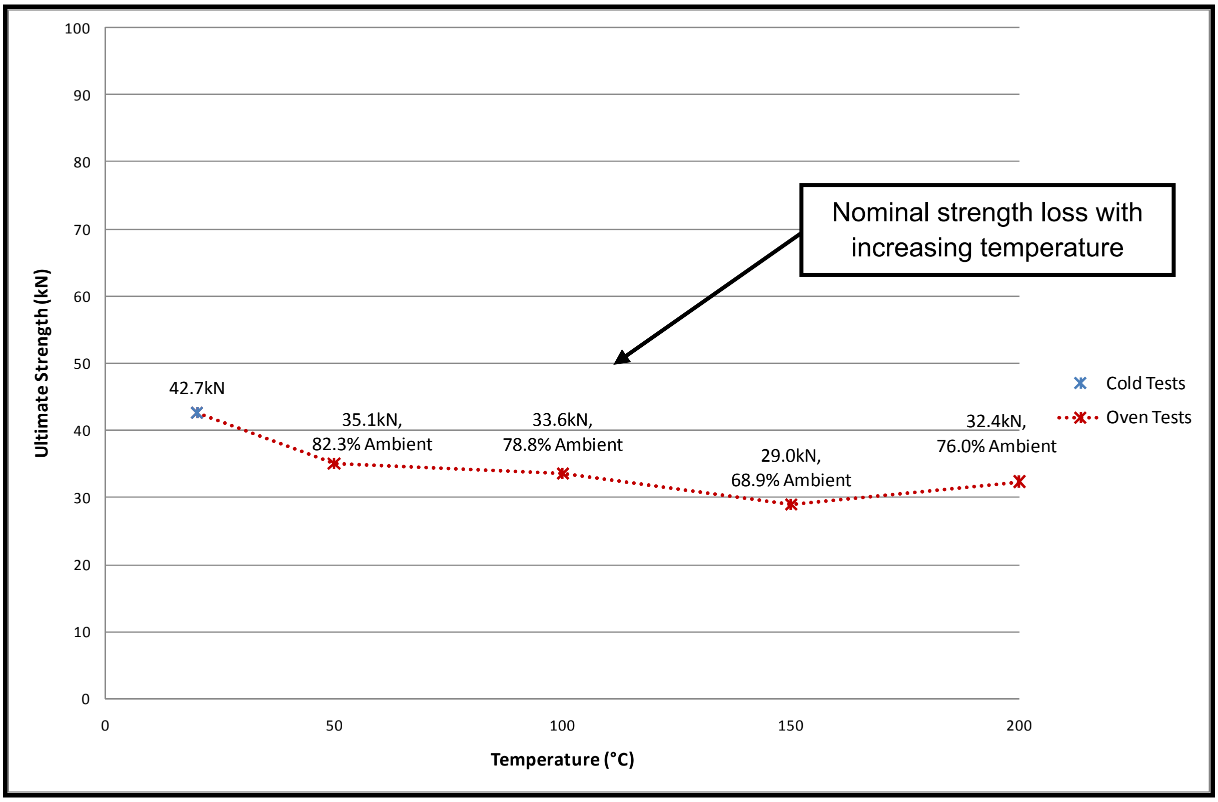

Figure 6-14 – Lagscrewbolt Cold and Oven Test Results

Observing the slope of data points indicates a general trend of strength decrease with increasing temperature, as seen in Figure 6-14. Oven test results suggest the strength of the connection reduces at a steady rate as the temperature increases. Furthermore, oven test results demonstrate that the Lagscrewbolt specimen maintains considerable strength at high temperatures. Additional testing could quantify the strength decrease with temperature.

At lower temperatures, ambient through 100°C, Lagscrewbolt test specimens exhibited the Mode 1 Failure: a tensile confinement failure consistent with cracking perpendicular to the laminations within the LVL section. Test specimens at 150°C and 200°C exhibited a Mode 5 Failure: pull out of the Lagscrewbolt product with significant amount of wood between the threads. This failure mechanism represented a shear failure in the LVL section occurring at the steel-wood interface where the LVL engaged the Lagscrewbolt threads. A summary of failure mechanisms for Lagscrewbolt test specimens subjected to oven testing is shown in Table 6-12:

Temperature	Failure Mode
20°C	1
50°C	1
100°C	1
150°C	5
200°C	5

Table 6-12 – Lagscrewbolt Cold and Oven Test Failure Modes

Analysis of failure modes with Lagscrewbolt test specimens suggests that as the temperature increases, the shear strength of the LVL parallel to the laminations experiences a greater strength loss than the strength of the LVL perpendicular to the laminations. Further testing with LVL could evaluate the strength and behaviour of NelsonPine LVL at high temperatures.

6.6. Comparison with Previous Testing

Comparing the experimental data with available literature provides a method for verifying oven test results. Available data on testing of steel to wood connections at high temperatures, however, remains modest. Previous works by Barber (1994) and Harris (2004) provide guidance on the behaviour of epoxy grouted steel threaded rods at high temperatures, with no available data for either the Timberlinx or Lagscrewbolt products for comparison.

6.6.1. Barber (1994)

Tensile pull out tests of epoxy grouted steel threaded rods at high temperatures were conducted by Barber (1994) to evaluate the heating effects on all-purpose epoxy connections. Two epoxies were used for testing, the West System ‘Z105 resin’ and ‘Z206 hardener’ and Nuplex ‘K80 Winter’ all-purpose epoxy. 20mm diameter steel threaded rods were embedded 200mm into a 90mm x 90mm Radiata Pine section for experimentation. Previous knowledge suggested temperatures above 100°C were outside the range of use for all-purpose epoxies. This assumption prompted oven testing between 20°C and 90°C at 10°C increments.

Testing results with the two epoxies, shown in Figure 6-15 reproduced from Barber (1994), demonstrate a distinct and significant drop with increase in temperature. Trend line data indicates the ultimate load values experience a decrease of nearly 80% in strength up to the maximum tested value of 90°C.

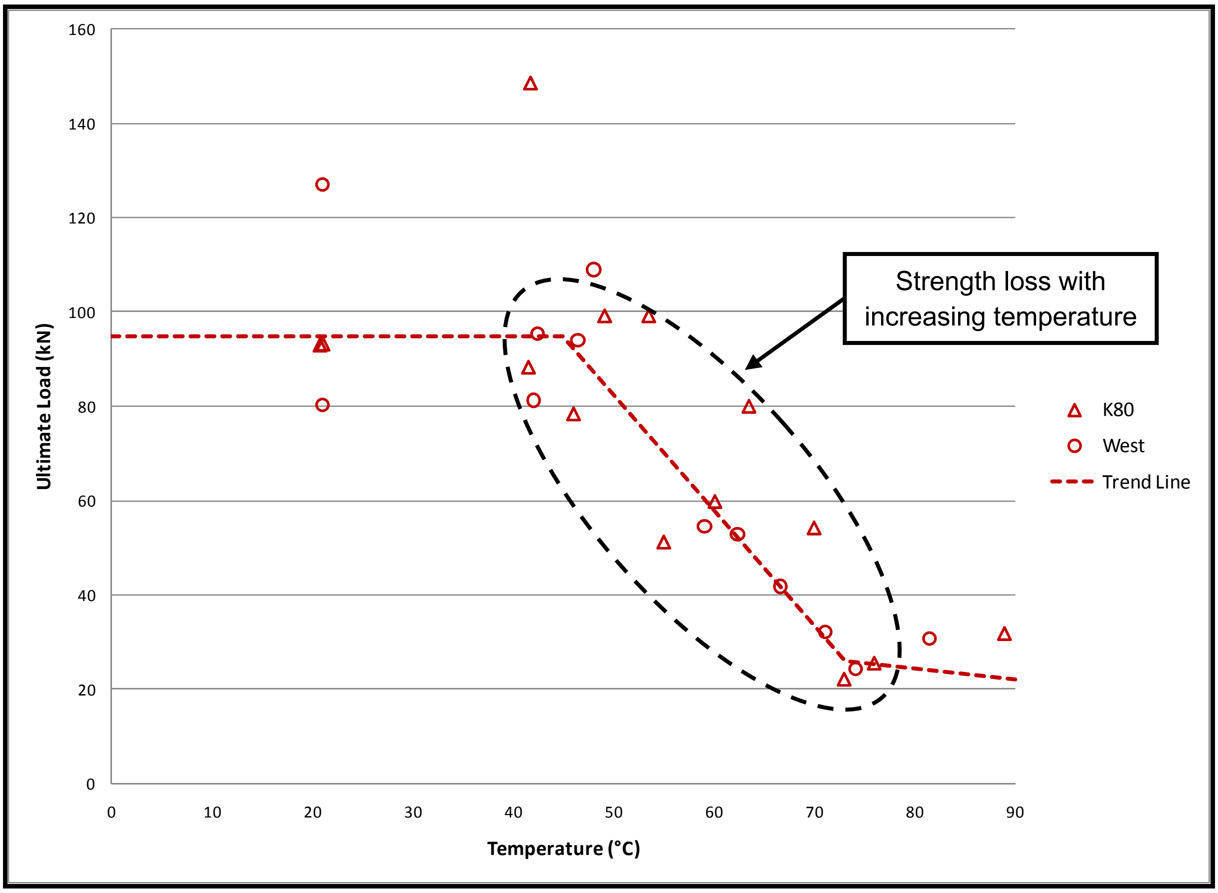

Figure 6-15 – Oven Tension Test Data (Modified from Barber, 1994)

When comparing oven testing data with previous oven tension tests by Barber (1994), epoxy behaviour appears consistent between both tests. A decrease in ultimate strength with an increase in temperature appears in both cases. Oven testing performance for high temperature and all-purpose epoxies is presented in Figure 6-16 by providing oven test ultimate strengths as a percentage of the ultimate strength at cold temperatures. Analysis suggests that all three high-temperature epoxies demonstrate reduced strength loss at high temperatures compared to previously tested all-purpose epoxies.

The Fischer high temperature epoxy displays considerably greater strength maintained at high temperatures when compared to previous epoxy test data. Comparing high temperature epoxy strength against trend line data provided by Barber (1994) suggests high temperature epoxy displays greater ultimate strength performance than all-purpose epoxies at high temperature conditions.

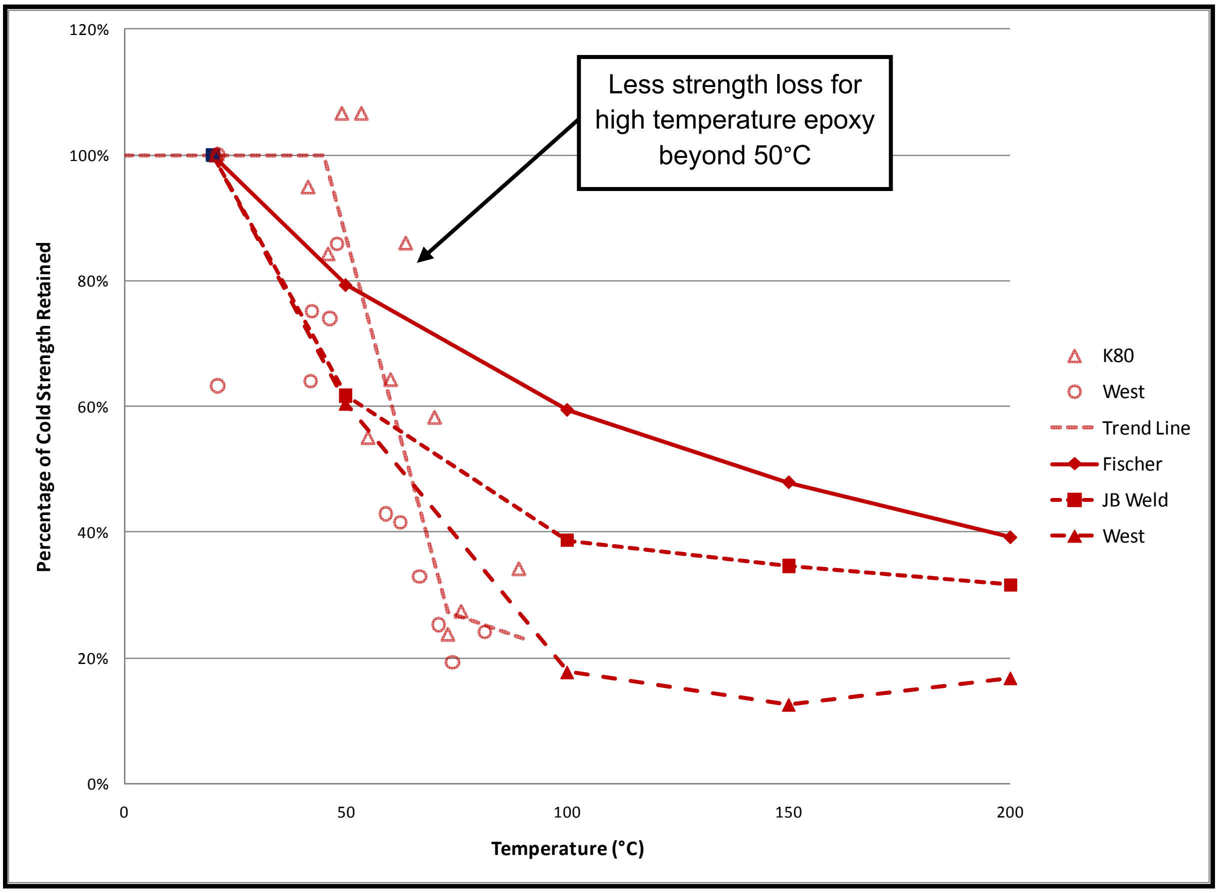

Figure 6-16 – Oven Test Results - Barber (1994) Comparison

6.6.2. Harris (2004)

Tensile pull out tests of epoxy grouted steel to wood connections under heated conditions were engaged by Harris (2004) to determine all-purpose epoxy behaviour at high temperatures. Three different epoxies were used for experimentation; each representing an all-purpose epoxy adhesive used in common construction practice.

Test specimens were tested in 63mm square and 105mm square sections of Carter Holt Harvey with 16mm diameter steel rod and 300mm embedment. Oven testing began at 50°C and proceeded at 25°C increments to the previously established limit of 100°C. Results from the testing were used to draw conclusions for all-purpose epoxy performance at high temperatures.

A similar trend is apparent when observing the test results presented in Figure 6-17. All three epoxies display a significant drop in strength with increase in temperature. While the ultimate strength for two of the three epoxies is significantly greater under cold conditions, each epoxy experienced a reduction of more than 60% in strength when tested at the 100°C plateau. The strength loss with temperature suggests that further testing beyond the 100°C mark would demonstrate continued strength loss until zero strength remained. Conclusions from Harris (2004) suggested that epoxy use beyond 100°C was not plausible due to the significant loss of strength.

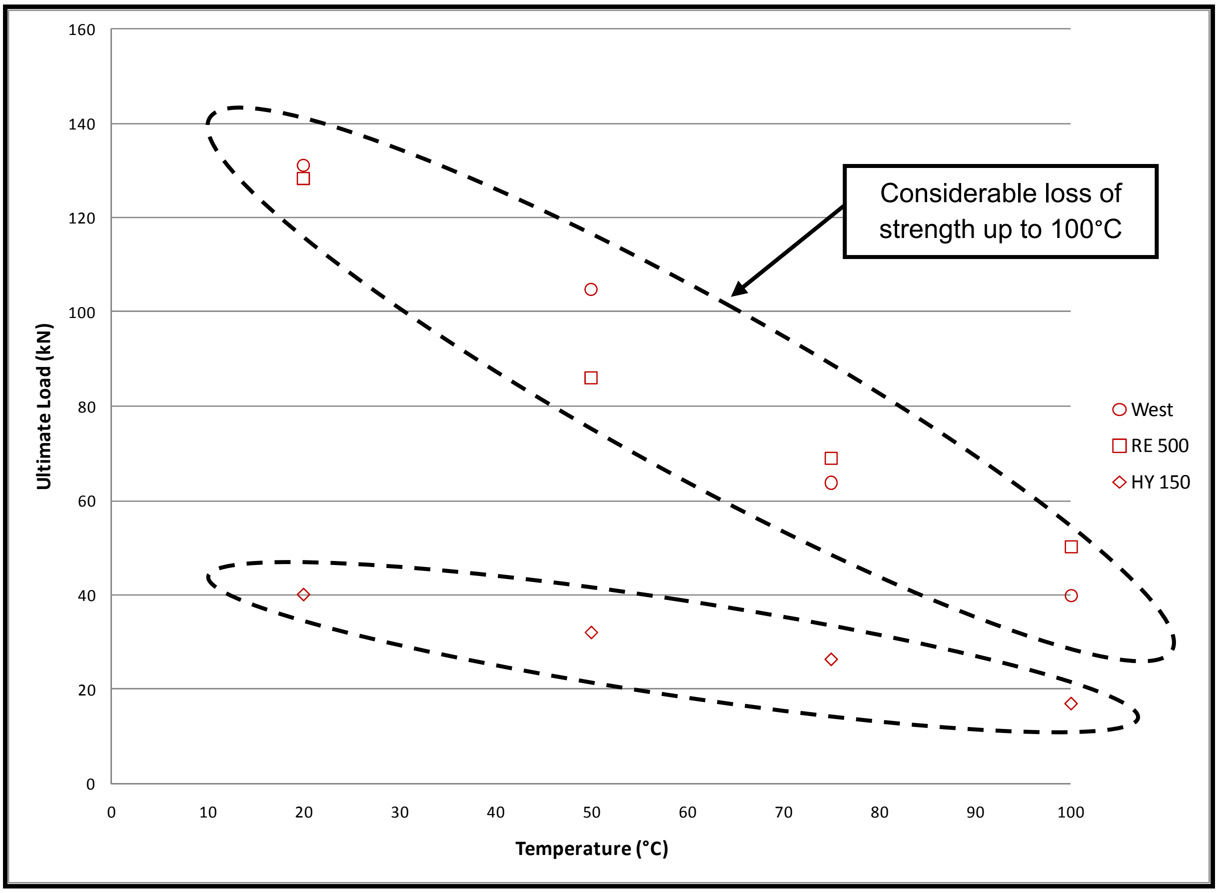

Figure 6-17 – Mean Oven Test Results (Harris, 2004)

A comparison between experimental oven testing data and Harris' (2004) mean oven test data is presented in Figure 6-18. Data points demonstrate the percentage of cold strength maintained for each epoxy tested at high temperatures. A comparison reveals that high temperature epoxies display similar strength loss with increase in temperature. While the JB Weld high temperature epoxy performance remains consistent with Harris' mean test data, the West epoxy falls slightly lower, depicting greater strength loss with temperature compared to all-purpose epoxy. The Fischer high temperature epoxy, however, displayed a greater ability to maintain strength (as a percentage of the ultimate strength at ambient conditions) when compared to both all-purpose and other high temperature epoxies.

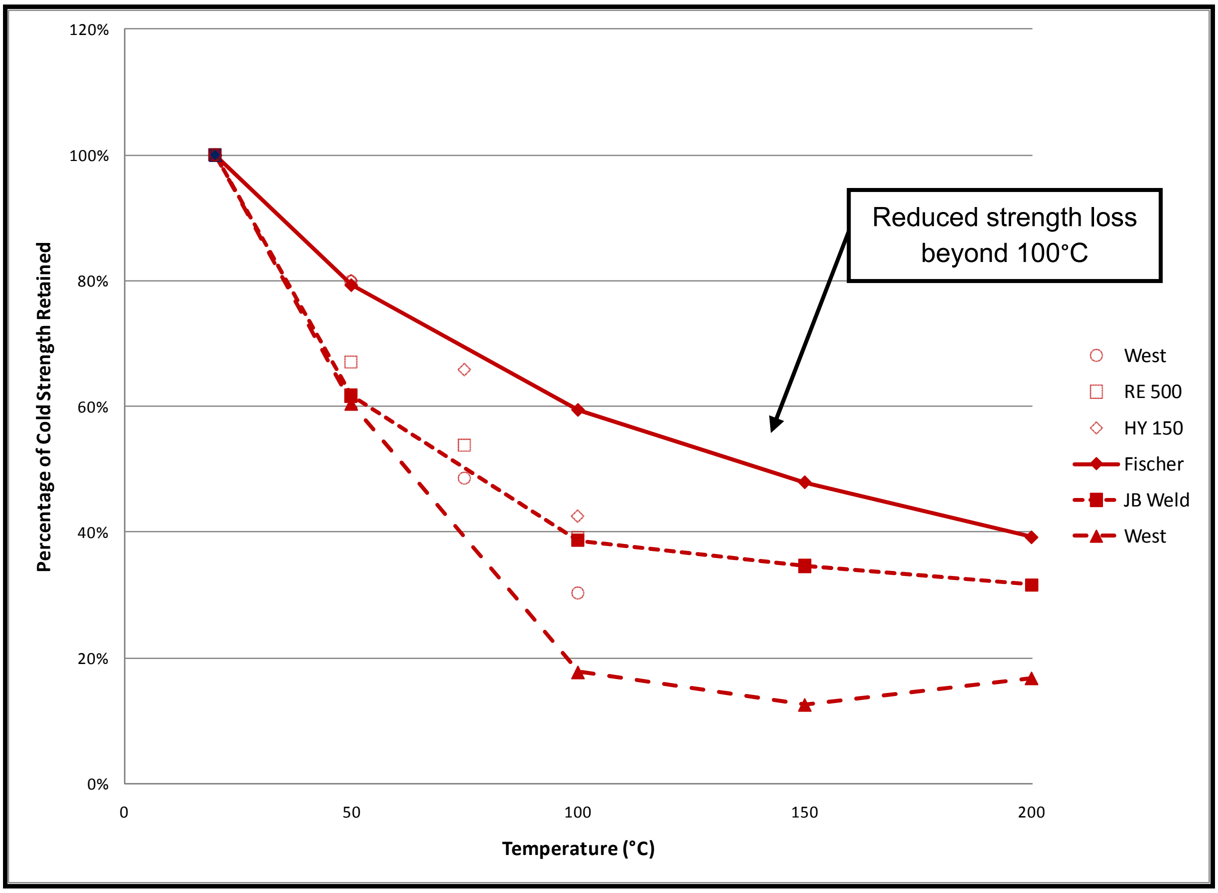

Figure 6-18 – Oven Test Results - Harris (2004) Comparison

6.7. Conclusions

Experimental oven testing was performed to determine the behaviour and performance of steel to wood connections at high temperatures. Epoxy grouted steel threaded rods and proprietary mechanical fasteners were heated to temperatures ranging from 50°C to 200°C at 50°C increments, and subjected to tensile testing. Oven testing isolated the temperature effect and removed the variable effects of charring and pyrolysis associated with wood combustion.

Comparing oven test results with cold test results and available literature provided insight into the behaviour and performance of connections at high temperatures. Testing verified an ultimate strength decrease with an increase in temperature. In addition, oven testing enhanced the understanding of rates of strength loss with temperature. Results suggest steel to wood connections may have greater strength beyond the 100°C strength plateau than previously anticipated.

6.7.1. Epoxy Specimens

Previous testing with epoxies revealed a considerable decrease in strength with increase in temperature (Barber, 1994 and Harris, 2004). Research suggested that epoxy adhesive at high temperatures may not be useful due to significant strength loss beyond 100°C (Barber, 1994). Experimental testing confirmed this reduced strength phenomenon. However, testing with high temperature epoxies demonstrated favourable strength behaviour beyond 100°C. Oven testing with high temperature epoxy has suggested that additional strength capacity may be maintained beyond 100°C.

Comparing ultimate load values for cold and oven testing revealed a significant initial strength decrease to 100°C. Specimens tested beyond 100°C displayed a less rapid strength decrease with increasing temperature.

Oven testing revealed a trend for the test specimen failure mechanism. At lower temperatures, the dominant failure mode was a brittle failure in the LVL. At high temperatures, the adhesive strength of the epoxy at the epoxy-wood interface became the dominant failure mechanism. Exposing high temperature epoxy specimens to high temperature causes a decrease in adhesive strength, resulting in failure at the epoxy-wood interface.

Oven testing was performed to determine the qualitative behaviour of high temperature epoxy at elevated temperatures. Test results indicate that steel to wood connections demonstrate considerable strength beyond the 100°C strength plateau. Additional testing would be necessary to enhance understanding on a more quantitative level. Future research could provide definitive equations that may predict expected strength and behaviour for the use of high temperature epoxies at elevated temperatures.

6.7.2. Timberlinx Specimens

The Timberlinx 'A475' steel to wood connector displayed a strength decrease with increasing temperature. Oven test specimens displayed brittle wood failures, similar to cold testing. Test results with Timberlinx test specimens have expanded the understanding of this innovative connection type at high temperatures, as previous testing data provided information for tensile tests at ambient conditions only.

Comparing ultimate strength results at ambient and high temperatures demonstrated that the Timberlinx steel to wood connector sustained considerable strength at high temperatures. Despite a fluctuation in strength values, the overall trend revealed gradual strength decrease with increasing temperature.

Analyzing the failure mechanism for Timberlinx test specimens displayed the consistency in failure modes, with wood failures occurring in all test samples. Increases in temperature decreased the ultimate strength of the connection but did not alter the failure mechanism.

Oven testing was performed to evaluate the strength and behaviour for the Timberlinx 'A475' steel to wood connector at high temperatures. The strength comparison with cold test results suggests that the connector maintains a considerable amount of strength at high temperatures. Additional testing of Timberlinx connections could provide methods for the quantitative analysis of connection strength with increased temperature.

6.7.3. Lagscrewbolt Specimens

Lagscrewbolt test specimens displayed a steady decrease in ultimate strength with increase in temperature. Failure modes for tested specimens provided surprising behaviour, evidenced by varying failure methods at increasing temperatures.

A comparison of cold and oven test data with Lagscrewbolt samples revealed a considerable amount of strength maintained at high temperatures. Results suggest that Lagscrewbolt specimens experienced gradual and steady strength loss when exposed to high temperatures. In fact, the maximum strength loss in Lagscrewbolt specimens was only 40%.

A failure mode analysis revealed surprising behaviour. Lagscrewbolt specimens experienced a transition in failure mode between 100°C and 150°C. Specimens transitioned from confinement failures at lower temperatures to shear failures at higher temperatures. Results indicate that the strength of LVL in shear parallel to grain decreased at a faster rate than tension perpendicular to grain. Future testing could evaluate this behaviour and provide further research into temperature effects on LVL strength.

Tensile testing of Lagscrewbolt products evaluated the behaviour and performance of Lagscrewbolt connections at high temperatures. Lagscrewbolt test specimens demonstrated minimal strength loss with temperature, yet revealed surprising failure methods with LVL strength at high temperatures. Additional testing could clarify issues and concerns raised by Lagscrewbolt oven testing and seek to improve the understanding of this steel to wood connector.

7. Cooled Testing

The third phase of experimentation involved cooled testing of steel to wood connections. Cooled testing was performed to determine the residual ultimate strength of epoxy connections subjected to heating and allowed to cool to ambient temperature. This procedure simulated the exposure to a minor fire event, where the connection is subjected to heat and hot gasses without demonstrating any charring. Both ultimate strengths and failure modes were compared with results from cold and oven testing to determine the performance and behaviour of steel to wood connections exposed to minor fire conditions.

7.1. Cooled Test Specimens

Cooled test specimens were constructed the same as cold and oven test specimens. Epoxy specimens were the only specimens tested, as both the Timberlinx and Lagscrewbolt proprietary steel products were expected to return to full strength when heated and allowed to cool to ambient temperatures.

7.1.1. Epoxy Specimens

Epoxy specimens used for cooled testing were identical to specimens used in cold and oven testing. Additional information on cooled test specimens can be found in Section 5.1.1. Diagrams of epoxy specimens used in oven testing are shown in Figure 5-1 and Figure 5-2 in the Cold Testing section.

7.2. Testing Procedure

Cooled testing was performed with the Avery Testing Machine in the University of Canterbury Civil Engineering Laboratory to determine the ultimate strength for epoxy specimens that had been heated to elevated temperatures and allowed to cool to ambient.

7.2.1. Oven Heating

Epoxy test specimens were heated in the Structures Extension Laboratory oven overnight to provide consistent heating throughout the entire specimen. Cooled test specimens were heated to temperatures ranging from 50°C to 200°C, at 50°C increments, in accordance with the previous oven testing protocol. Additional information regarding oven heating can be found in Section 6.2.1.

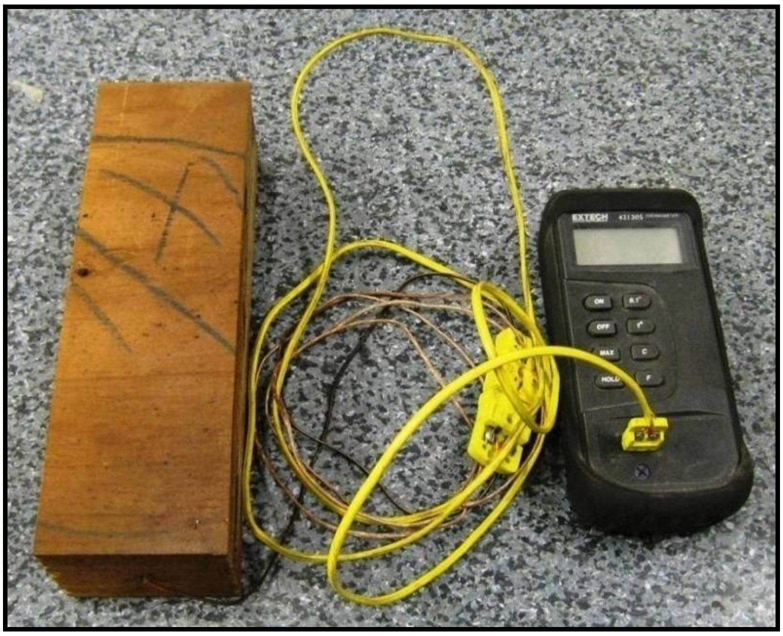

Figure 7-1 – Digital Thermometer

A 105mm square section of NelsonPine LVL with an embedded thermocouple was attached to a digital thermometer to observe interior specimen temperatures. This section (shown in Figure 7-1) was placed in the oven with cooled test specimens to verify consistent heating throughout the specimen. When the digital thermometer confirmed the interior LVL temperature reached the specified testing temperature, specimens were removed from the oven and allowed to cool to ambient conditions prior to testing.

7.2.2. Cooling Time

Test specimens were allowed to cool overnight before engaging in tensile testing. Testing was performed once the digital thermometer confirmed that the interior specimen temperature had returned to ambient conditions.

7.2.3. Specimen Preparation

Cooled test specimens were prepared in exactly the same process as those undergoing cold testing. This included adding the custom steel bracket and loading the specimen in the Avery Testing Machine. Additional information on specimen preparation can be found in Section 5.2.1.

7.2.4. Load Application

Tensile load was applied to test specimens in the Avery Testing Machine located in the University of Canterbury Civil Engineering Laboratory. Load application for cooled test specimens proceeded in the same method as cold and oven tests. Additional information regarding load application can be found in Section 5.2.2.

7.2.5. Data Recording

Output from load cells and gages in the Avery Testing Machine during tensile testing was recorded by the Universal Data Logger (UDL) digital software attached to the University of Canterbury Civil Engineering Laboratory computer. Additional information regarding data recording can be found in Section 5.2.3.

7.2.6. Specimen Failure

Specimen failure for cooled tests was defined the same as for cold and oven tests. Failure was evaluated purely in the strength domain and defined as the point at which the steel to wood connection could no longer sustain any additional tensile load. Further information regarding specimen failure can be found in Section 5.2.4.

7.3. Results

Tensile testing of cooled test specimens was performed to evaluate the residual strength of epoxy grouted steel threaded rods subjected to heating and allowed to cool to ambient temperature, simulating a minor fire event.

Ultimate strength values for the epoxy grouted steel threaded rod test specimens are shown found in Table 7-1 and Figure 7-2 with failure modes presented in Table 7-2:

Temperature	Fischer	JB Weld	West
50°C	63.1	66.4	71.5
100°C	60.5	72.9	70.1
150°C	65.7	64.0	74.6
200°C	59.8	63.7	65.1

Table 7-1 – Cooled Test Ultimate Strength Results (kN)

Temperature	Fischer	JB Weld	West
50°C	1	1	1
100°C	3	3	3
150°C	3	3	3
200°C	3	3	3

Table 7-2 – Cooled Test Failure Mode Results

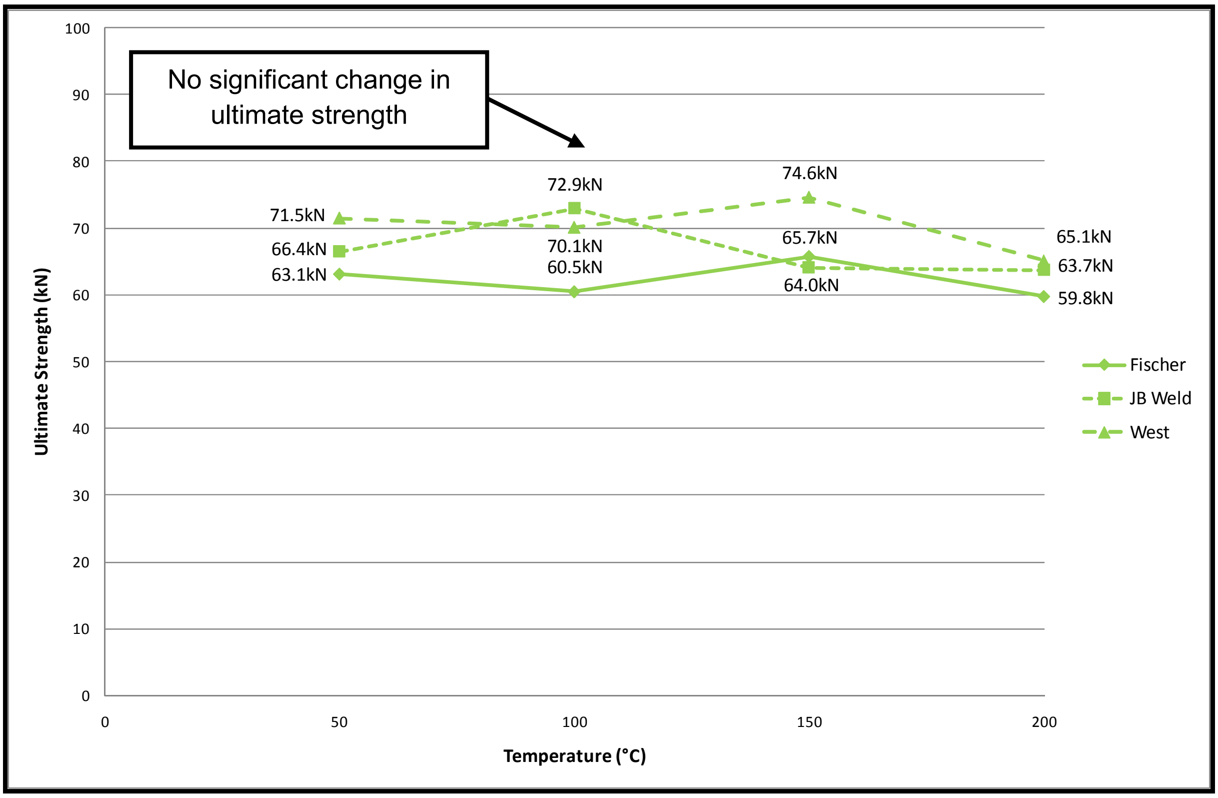

Figure 7-2 - Cooled Test Results

Ultimate strengths for cooled tests, shown in Figure 7-2, suggest that ultimate strength values for high temperature epoxies remain consistent regardless of the temperature to which the specimen was heated. Testing results indicate that epoxy grouted steel threaded rod specimens demonstrate nominal, if any, strength loss when cooled to ambient temperature.

Failure modes for cooled test specimens demonstrated consistency for all three high temperature epoxies subjected to tensile testing. At lower temperatures, specimens demonstrated Mode 1 Failures. Above 50°C, specimens exhibited Mode 3 Failures. Additional information for Failure Mode 1 and Failure Mode 3 can be found in Section 5.4.1 and Section 5.4.3, respectively.

7.4. Failure Modes

Tensile testing of cooled test specimens displayed two distinct failure modes. Both Mode 1 and Mode 3 Failures were first witnessed in cold testing and reappeared in cooled test specimens. Each failure mechanism represents a brittle failure mode occurring in the LVL.

7.4.1. Mode 1 Failure

Mode 1 Failures were witnessed with cooled test specimens heated to 50°C. The Mode 1 Failure, shown in Figure 7-3, represents a brittle confinement failure within the LVL with cracking perpendicular to the laminations. Additional information on Mode 1 Failures can be found in Section 5.4.1.

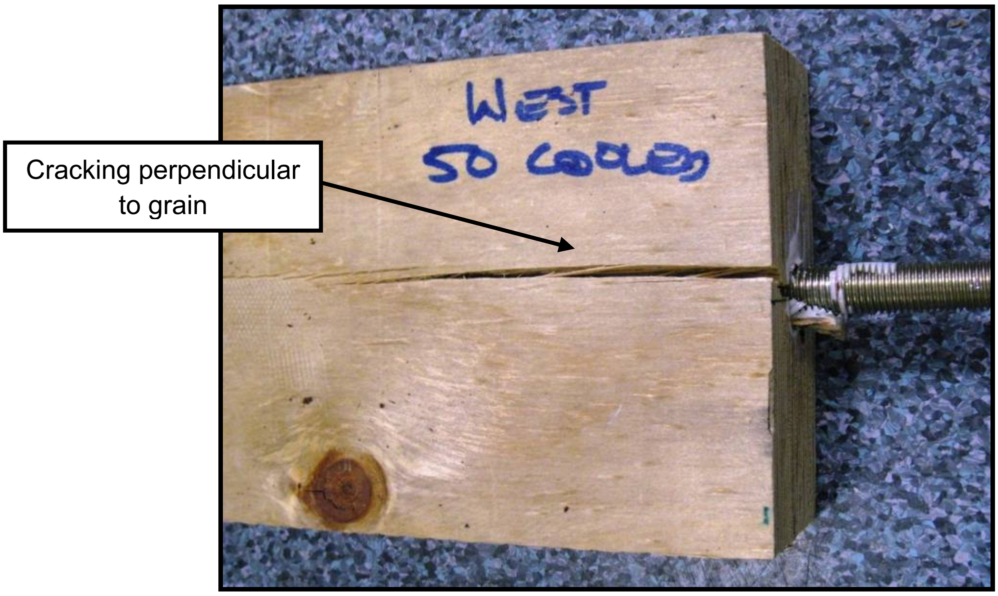

Figure 7-3 – Mode 1 Failure in West 50°C Cooled Test Specimen

7.4.2. Mode 3 Failure

The Mode 3 Failure can be classified as a brittle pull out failure occurring in the LVL. As tensile forces increased, the epoxy grouted steel threaded rod pulled out from the LVL section, removing a significant amount of attached wood. More information on Mode 3 Failures can be found in Section 5.4.3. The Mode 3 Failure (shown in Figure 7-4) was witnessed in all cooled test specimens heated to 100°C and above.

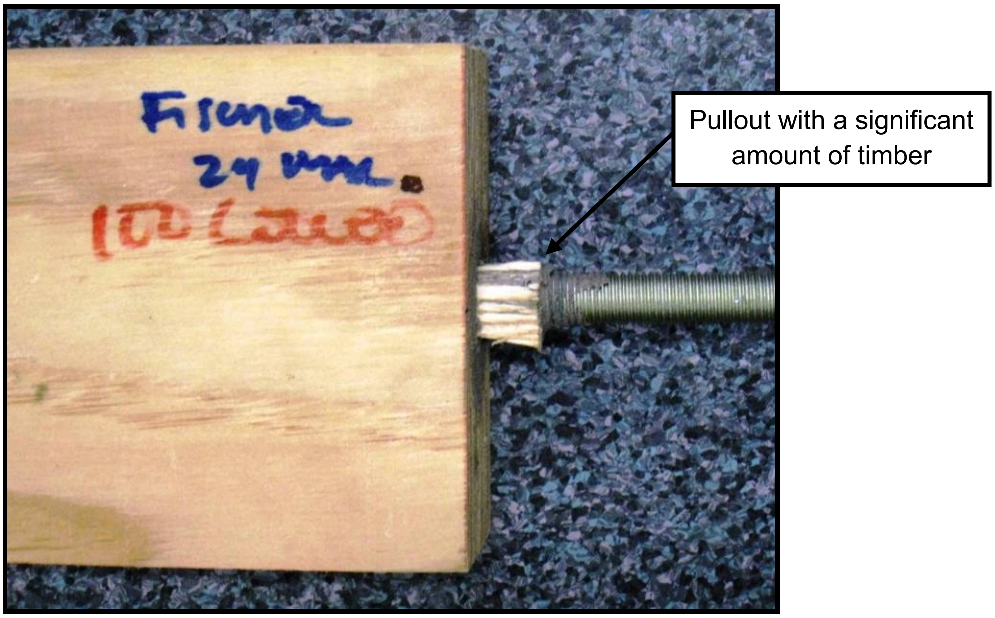

Figure 7-4 – Mode 3 Failure in Fischer 100°C Cooled Test Specimen

7.5. Comparison with Cold Testing

Experimental cooled testing was performed to evaluate the high temperature epoxy strength following a minor fire event. Test results from cooled tests can be used for comparison with cold test results to evaluate the strength loss for epoxy exposed to high temperatures.

7.5.1. Epoxy Specimens

All three high temperature epoxies were used for cooled test specimens. These included the Fischer, JB Weld and West epoxies for constructing epoxy grouted steel threaded rod specimens.

7.5.1.1. Fischer 'FIS V 360 S' Injection Mortar

Test specimens using the Fischer 'FIS V 360 S' high temperature epoxy appear to gain strength when heated to high temperatures and allowed to cool. Test results for cooled test specimens subjected to tensile testing are shown in Table 7-3 and Figure 7-8 with a summary of failure modes provided in Table 7-10:

Temperature	Ultimate Load (kN)	Ultimate Load / Ambient Ultimate Load (%)
20°C	60.3	100
50°C	63.1	104.6
100°C	60.5	100.3
150°C	65.7	108.9
200°C	59.8	99.2

Table 7-3 – Fischer Cold and Cooled Test Results

When exposed to high temperatures and allowed to cool to ambient conditions, epoxy grouted steel threaded rod specimens using the Fischer epoxy demonstrated increased ultimate strength compared to the strength obtained at cold temperature alone. The heating and cooling process increased the ultimate strength to nearly 110% of the ultimate strength at ambient temperature.

The rate of strength increase is shown in Figure 7-5. Test specimens heated from 50°C to 200°C and allowed to cool displayed consistent, increased ultimate strength when compared to the strength at cold temperature. The heating and cooling process appeared to moderately increase the ultimate strength of the connection. Further testing of cooled test specimens using Fischer epoxy could validate this strength increase.

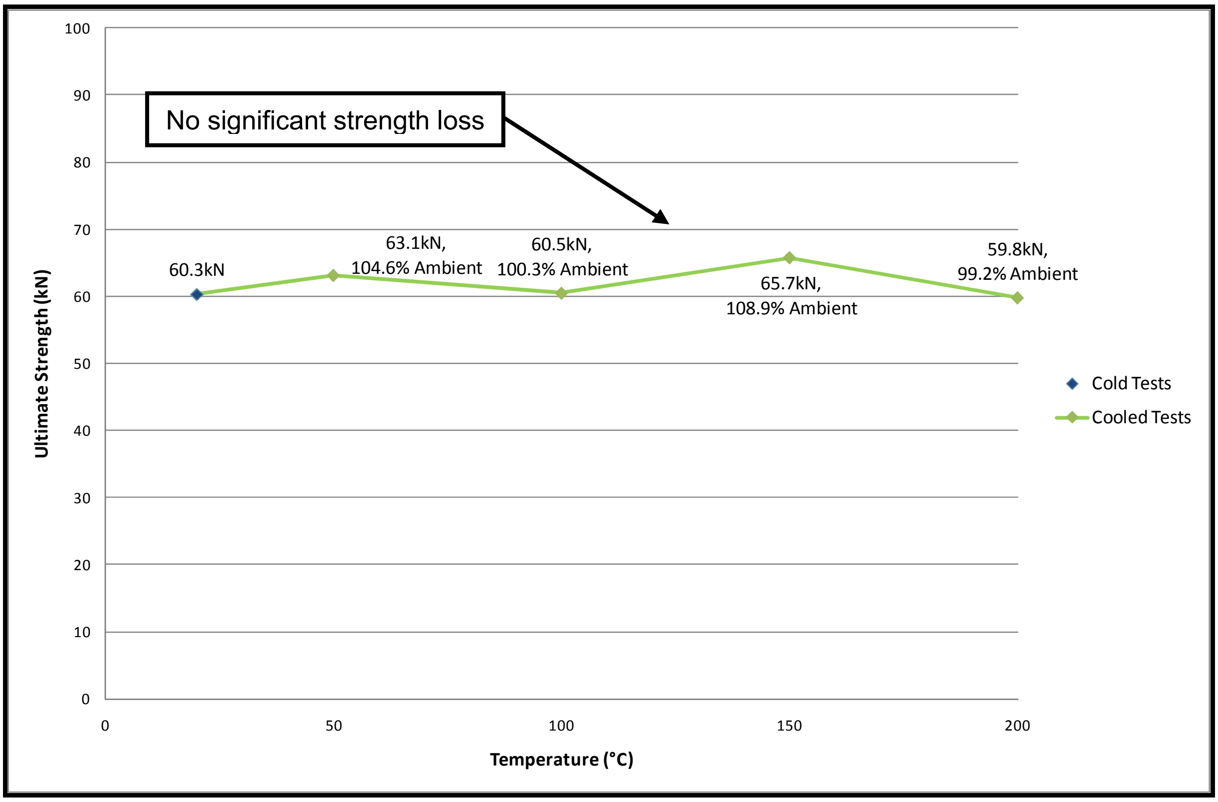

Figure 7-5 – Fischer Cold and Cooled Test Results

A failure mode analysis for Fischer cooled test specimens displayed a change of failure mechanism when heated beyond 50°C. Both the cold and 50°C test specimens exhibited a Mode 1 Failure while remaining specimens demonstrated Mode 3 Failures. Additional information on Failure Mode 1 and Failure Mode 3 can be found in Section 5.4.1 and Section 5.4.3 respectively.

Temperature	Failure Mode
20°C	1
50°C	1
100°C	3
150°C	3
200°C	3

Table 7-4 – Fischer Cold and Cooled Test Failure Modes

Comparing failure modes suggests that heating and cooling test specimens caused a transition in epoxy behaviour. Test specimens at lower temperatures exhibited Failure Mode 1: a confinement failure in the LVL. Specimens heated to high temperatures displayed Failure Mode 3: a tensile pull out failure in the LVL.

7.5.1.2. JB Weld 'Industro Weld'

Comparing cooled test results with cold test results for JB Weld specimens displayed no significant difference in ultimate strength values obtained through tensile testing. Cooled specimen ultimate strength values appeared to remain consistent regardless of heated temperature. Results are shown in Table 7-5 and Figure 7-6, with failure modes presented in Table 7-6:

Temperature	Ultimate Load (kN)	Ultimate Load / Ambient Ultimate Load (%)
20°C	66.8	100
50°C	66.4	99.3
100°C	72.9	109.1
150°C	64.0	95.8
200°C	63.7	95.4

Table 7-5 – JB Weld Cold and Cooled Test Results

Cooled test specimens using the JB Weld epoxy showed consistency in ultimate strength values. Ultimate loads varied by less than 10% when compared to ultimate strength at cold temperature. An increase in heated temperature appeared to have no significant effect on the ultimate strength value for JB Weld cooled test specimens.

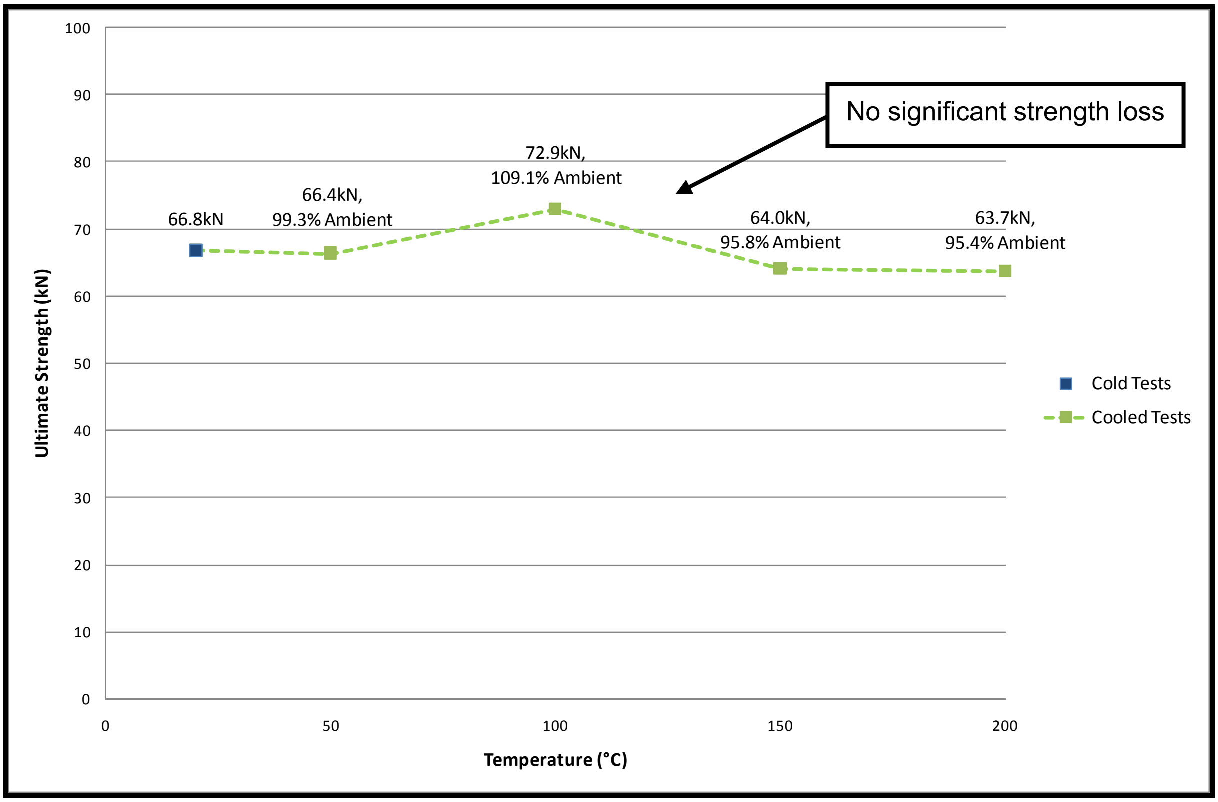

Figure 7-6 – JB Weld Cold and Cooled Test Results

Ultimate strength values displayed no significant changes in value with increases in temperature, as shown in Figure 7-6. Analyzing this constant behaviour suggests that the JB Weld epoxy demonstrates minimal strength loss when heated and allowed to cool following a minor fire event.

Temperature	Failure Mode
20°C	3
50°C	1
100°C	3
150°C	3

200°C	3

Table 7-6 – JB Weld Cold and Cooled Test Failure Modes

The failure mechanism for cooled JB Weld test specimens remained relatively consistent. Only the 50°C sample failed in an alternative manner. Most specimens experienced a Mode 3 Failure, in which the epoxy grouted steel threaded rod pulled out from the LVL section with a significant amount of wood attached. Additional information for Mode 3 failures can be found in Section 5.4.3.

The only specimen not to display the Mode 3 Failure, the 50°C sample, displayed a Mode 1 confinement failure during testing. Additional testing would be necessary to further understanding of JB Weld epoxy failure mode behaviour.

7.5.1.3. West System 'Z206' Epoxy Hardener and Adhesive Technologies 'ADR 310' Epoxy Resin

Cooled West epoxy specimens displayed a nominal amount of strength loss, but no significant changes in ultimate loads when compared to ambient temperatures. Values remained consistent except for a slight decrease in ultimate load for the 200°C test specimen. Ultimate load results for tensile testing of cooled specimens using the West epoxy are shown in Table 7-7 and Figure 7-7, with failure modes presented in Table 7-8:

Temperature	Ultimate Load (kN)	Ultimate Load / Ambient Ultimate Load (%)
20°C	74.3	100
50°C	71.5	96.3
100°C	70.1	94.3
150°C	74.6	100.4
200°C	65.1	87.6

Table 7-7 – West Cold and Cooled Test Results

West epoxy specimens demonstrated nominal strength loss following the heating and cooling process. Slight decreases in strength are visible for high temperatures, with a moderate strength loss displayed by the 200°C specimen. Ultimately, however, West epoxy maintained strength regardless of heated temperature.

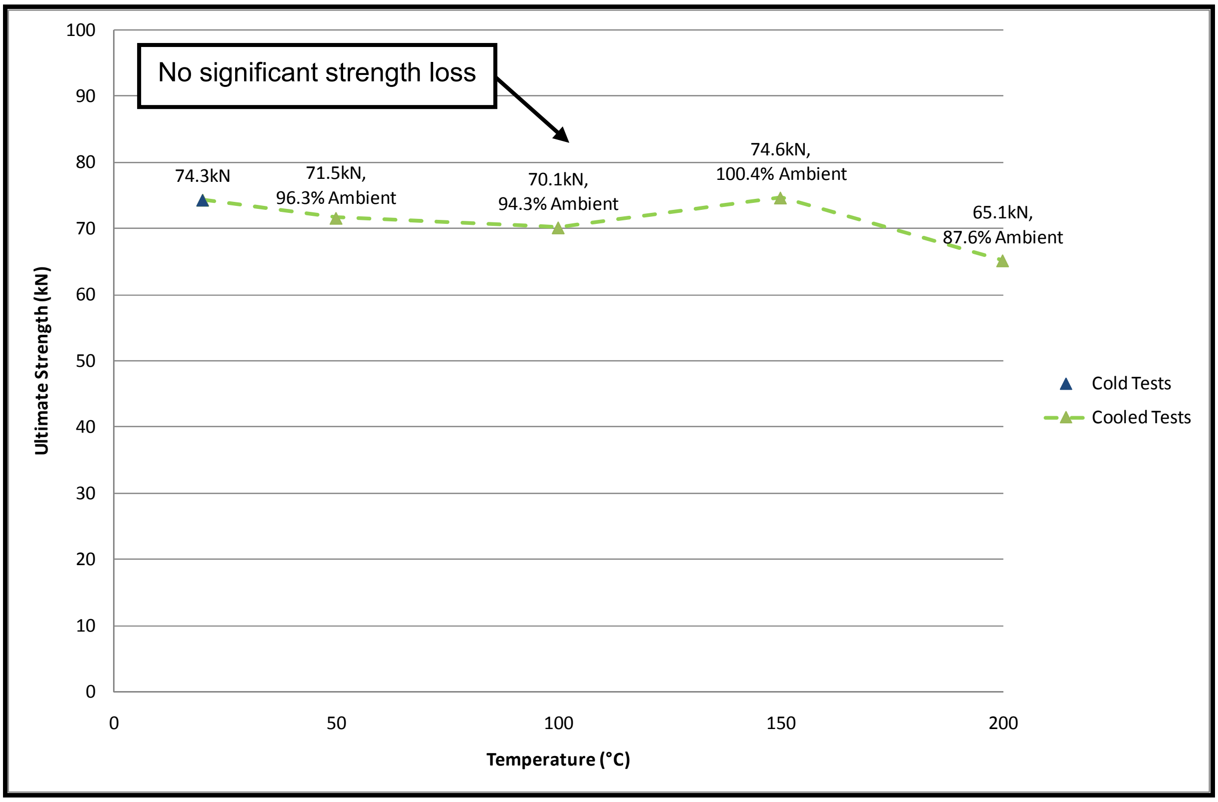

Figure 7-7 – West Cold and Cooled Test Results

The connection strength with increasing temperature, shown in Figure 7-7, remains relatively unchanged with no significant strength loss compared to cold conditions. The only noteworthy change in strength resulted in the 200°C test, which maintained nearly 90% of the ambient strength. Overall, the rate of strength change with increasing temperature appeared constant, suggesting a measure of reliability and consistency associated with cooled testing of West epoxy.

Temperature	Failure Mode
20°C	3
50°C	1
100°C	3
150°C	3
200°C	3

Table 7-8 – West Cold and Cooled Test Failure Modes

Failure mechanisms for the West epoxy behaved similar to the JB Weld specimens. Most samples displayed the Mode 3 Failure: pull out of the epoxy grouted steel threaded rod with a significant amount of timber attached. The 50°C sample, however, demonstrated a Mode 1 Failure: a confinement failure with splitting perpendicular to the laminations in the LVL.

The variety of failure modes suggests that the heating and cooling process caused changes in either the LVL or the West epoxy to result in an alternative failure mode. Further research could investigate this failure mode behaviour.

7.6. Comparison with Oven Testing

A comparison between cooled test results and oven test results establishes the difference in strength values resulting from the cooling process. The primary difference in cooled and oven testing was the temperature at which specimens were subjected to tensile testing, as cooled test specimens were allowed to return to ambient temperature prior to testing.

7.6.1. Epoxy Specimens

All three epoxy products were used for cooled testing of epoxy grouted steel threaded rods. These include the Fischer, JB Weld and West epoxy adhesive products.

7.6.1.1. Fischer 'FIS V 360 S' Injection Mortar

Oven testing with Fischer epoxy grouted steel threaded rod specimens at high temperature displayed reduced overall strength with increase in temperature. Compared to cooled specimens, ultimate load values for cooled tests appear to be consistently greater than oven values. This suggests that the cooling process allows the high temperature epoxy to recover strength with a decrease in temperature.

Test results for ultimate load are shown in Table 7-9 and Figure 7-8, with failure loads displayed in Table 7-10:

Temperature	Oven Test Ultimate Load (kN)	Cooled Test Ultimate Load (kN)
50°C	46.2	63.1
100°C	34.6	60.5
150°C	27.9	65.7
200°C	22.8	59.8

Table 7-9 – Fischer Oven and Cooled Test Results

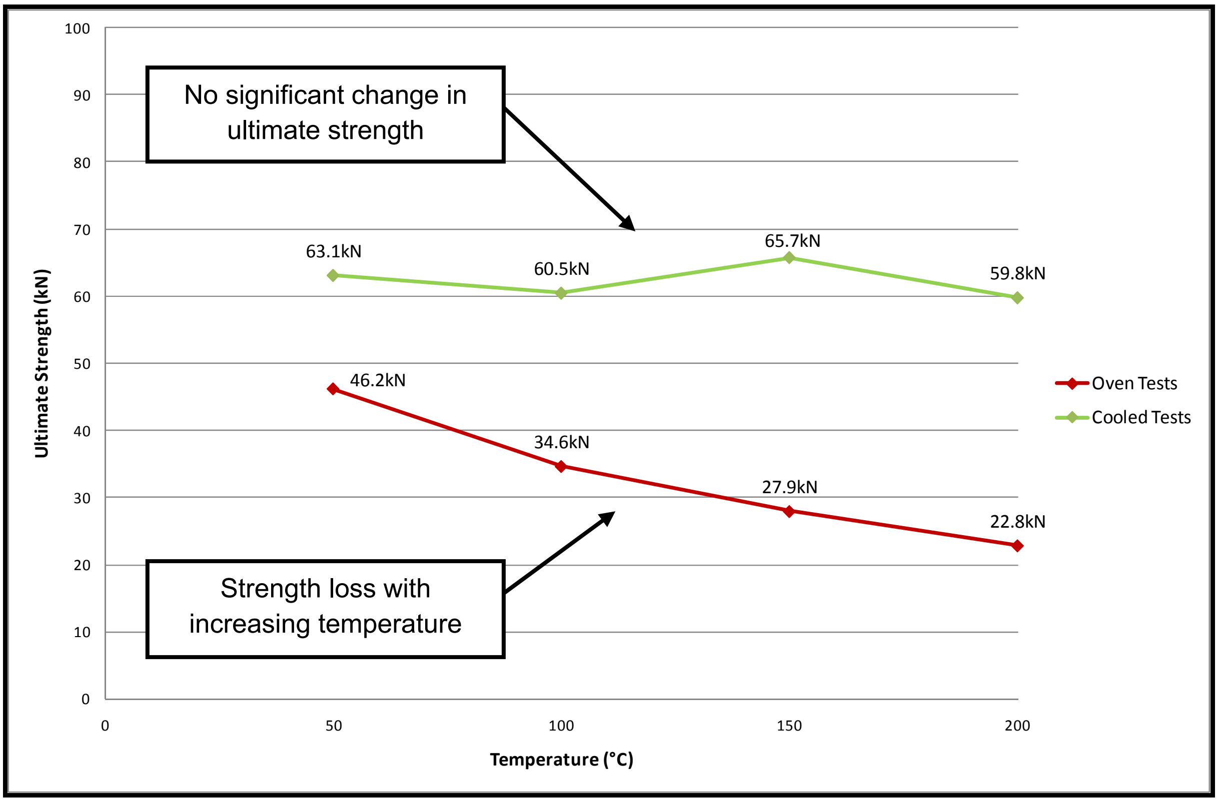

Figure 7-8 – Fischer Oven and Cooled Test Results

Observing the plot in Figure 7-8, it is apparent that ultimate strength values for cooled tests remain consistently greater than strength values for heated tests. As the temperature increases, ultimate strength for oven specimens decreases, whereas strength for cooled specimens remains relatively unchanged. Allowing specimens to cool from heated temperatures significantly improves the ultimate strength of the connection.

Fischer epoxy behaviour appears consistent for both oven and cooled test specimens. Specimens displayed confinement failures at lower temperatures with differing behaviour at high temperatures. Table 7-11 presents failure mode information:

Temperature	Failure Mode	
	Oven Tests	Cooled Tests
50°C	1	1
100°C	4	3
150°C	4	3
200°C	4	3

Table 7-10 – Fischer Oven and Cooled Test Failure Modes

At lower temperatures, test specimens displayed Mode 1 Failures: brittle confinement failures in the LVL marked by splitting perpendicular to the laminations. At high temperatures, specimen behaviour varies. When tested at high temperatures, specimens displayed the Mode 4 Failure: an epoxy failure with epoxy grouted steel threaded rod pull out at the epoxy-wood interface and the epoxy turning powdery and losing adhesive strength. Cooled test specimens displayed Mode 3 Failures: brittle wood failures with pull out of the epoxy grouted steel threaded rod with a significant amount of timber attached. Additional information on Mode 1 Failures, Mode 4 Failures and Mode 3 Failures can be found in Section 5.4.1, Section 6.4.3, and Section 5.4.3, respectively.

Contrasting the different failure mechanisms suggests that the Fischer epoxy lost adhesive strength at high temperature, evidenced by the consistent Mode 4 epoxy failures. At high temperatures, the epoxy weakened and became the dominant failure mechanism. When allowed to cool, however, the change in failure mechanism to a wood failure suggests that epoxy regained a considerable amount of strength. A strength increase in the epoxy forced the failure mechanism back into the wood. Additional testing could quantify the strength gain associated with cooling.

7.6.1.2. JB Weld 'Industro Weld'

Ultimate loads for JB Weld cooled tests remained markedly higher than those for oven tests. Results from tensile testing of epoxy grouted steel threaded rods using the JB Weld epoxy are shown in Table 7-11 and Figure 7-9, with failure modes resulting from tensile testing displayed in Table 7-12:

Temperature	Oven Test Ultimate Load (kN)	Cooled Test Ultimate Load (kN)
50°C	41.2	66.4
100°C	25.9	72.9
150°C	23.1	64.0
200°C	21.1	63.7

Table 7-11 – JB Weld Oven and Cooled Test Results

Ultimate strength values for oven and cooled tests displayed significant differences in strength. Ultimate failure loads demonstrated considerably greater strength for cooled JB Weld specimens compared to oven specimens. At high temperatures, the ultimate strength for cooled tests was as much as three times greater than oven tests.

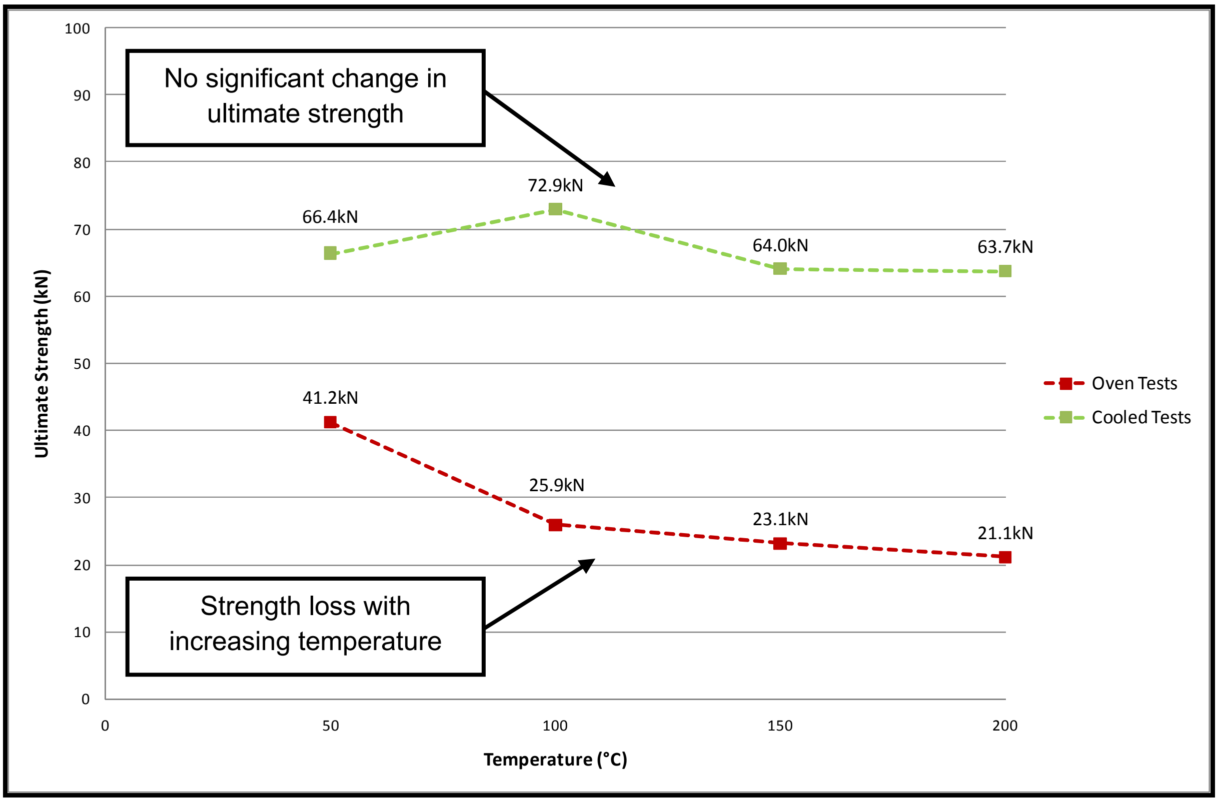

Figure 7-9 – JB Weld Oven and Cooled Test Results

As the temperature increased, ultimate strength values for JB Weld oven specimens decreased, while the ultimate strength for cooled specimens remained predominantly constant.

Failure methods for JB Weld test specimens behaved identical to Fischer test specimens. Mode 1 Failures were witnessed at lower temperatures for both oven and cooled tests, while oven tests displayed Mode 4 Failures and cooled tests displayed Mode 3 Failures. Results for failure modes of JB Weld oven and cooled test specimens are shown in Table 7-12:

Temperature	Failure Mode	
	Oven Tests	Cooled Tests
50°C	1	1
100°C	4	3
150°C	4	3
200°C	4	3

Table 7-12 – JB Weld Oven and Cooled Test Results

The heating and cooling process appears to cause a change in epoxy grouted steel threaded rod connection behaviour. At elevated temperatures JB Weld specimens transitioned from a Mode 4 epoxy adhesive failure to a Mode 3 wood failure.

Connection strength and performance for JB Weld test specimens behaved very similar to that for the Fischer epoxy. Connection strength was greatly improved by allowing the connection to return to ambient temperature.

7.6.1.3. West System 'Z206' Epoxy Hardener and Adhesive Technologies 'ADR 310' Epoxy Resin

Ultimate strength values for West cooled specimens remained significantly greater than values for oven testing. Results from cooled testing of West specimens are shown in Table 7-13 and Figure 7-10, with failure modes presented in Table 7-14:

Temperature	Oven Test Ultimate Load (kN)	Cooled Test Ultimate Load (kN)
50°C	44.9	71.5
100°C	13.2	70.1
150°C	9.4	74.6
200°C	12.5	65.1

Table 7-13 – West Oven and Cooled Test Results

West cooled specimens maintained strength after the heating and cooling process. Comparing oven test results with cooled test results demonstrated a significant difference in magnitude for both testing protocols. Differences in oven and cooled testing values are due to the considerable strength loss associated with tensile testing at high temperatures.

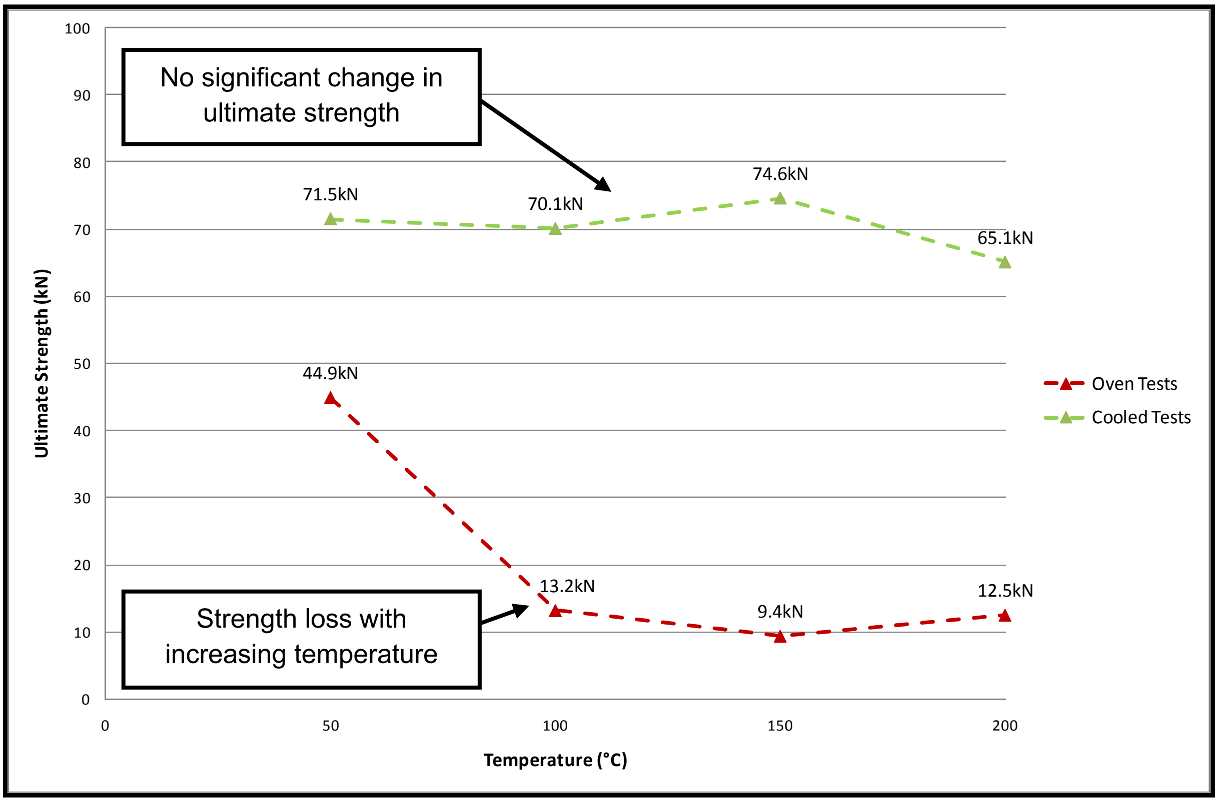

Figure 7-10 – West Oven and Cooled Test Results

West epoxy displayed the most significant strength loss with increasing temperature for oven specimens, resulting in the considerable difference in scale compared to ultimate strength of cooled test specimens.

Experimental samples using West epoxy displayed Mode 1 Failures at 50°C, with the remaining oven test specimens exhibiting Failure Mode 4 and cooled test specimens demonstrating Failure Mode 3. Failure modes for West oven and cooled specimens are shown in Table 7-14:

Temperature	Failure Mode	
	Oven Tests	Cooled Tests
50°C	1	1
100°C	4	3
150°C	4	3
200°C	4	3

Table 7-14 – West Oven and Cooled Test Results

West epoxy displayed the same transition in failure mode as the Fischer and JB Weld epoxies. Test specimens at lower temperatures displayed Mode 1 Failures, while oven specimens displayed Mode 4 Failures, and cooled specimens displayed Mode 3 Failures.

The ultimate load for cooled test specimens remained significantly greater compared to oven tests results. Cooled tests results suggest that the heating and cooling process does not significantly adversely affect the ultimate strength of the connection.

7.7. Comparison with Previous Testing

A comparison with previous tests of cooled specimens provided test data for which experimental results can be verified. Research performed by Harris (2004) involved tensile testing of cooled test specimens using steel threaded rods with all-purpose epoxy. Results from Harris suggest that cooled test specimens display no significant strength loss when compared to samples tested at ambient temperature.

7.7.1. Harris (2004)

Experimentation performed by Harris (2004) involved the use of all-purpose epoxy grout with 16mm diameter steel threaded rods, embedded 300mm into 63mm square and 105mm square Carter Holt Harvey sections. Test specimens were heated overnight to temperatures of 50°C, 75°C and 100°C and allowed to cool to ambient temperature prior to testing.

The heating and cooling regimen was intended to replicate the effects of a minor fire on structural elements. In a minor fire, elements are commonly exposed to high temperatures that result in minimal and/or insignificant charring or pyrolysis. Once the fire has been contained, elements return to ambient conditions. Cooled tests seek to clarify the heating effects these minor fires have on structural epoxy.

Results from testing performed by Harris suggest that epoxy specimens demonstrate no significant loss in strength when heated to high temperatures. Results from previous research by Harris (2004) are displayed in Figure 7-11:

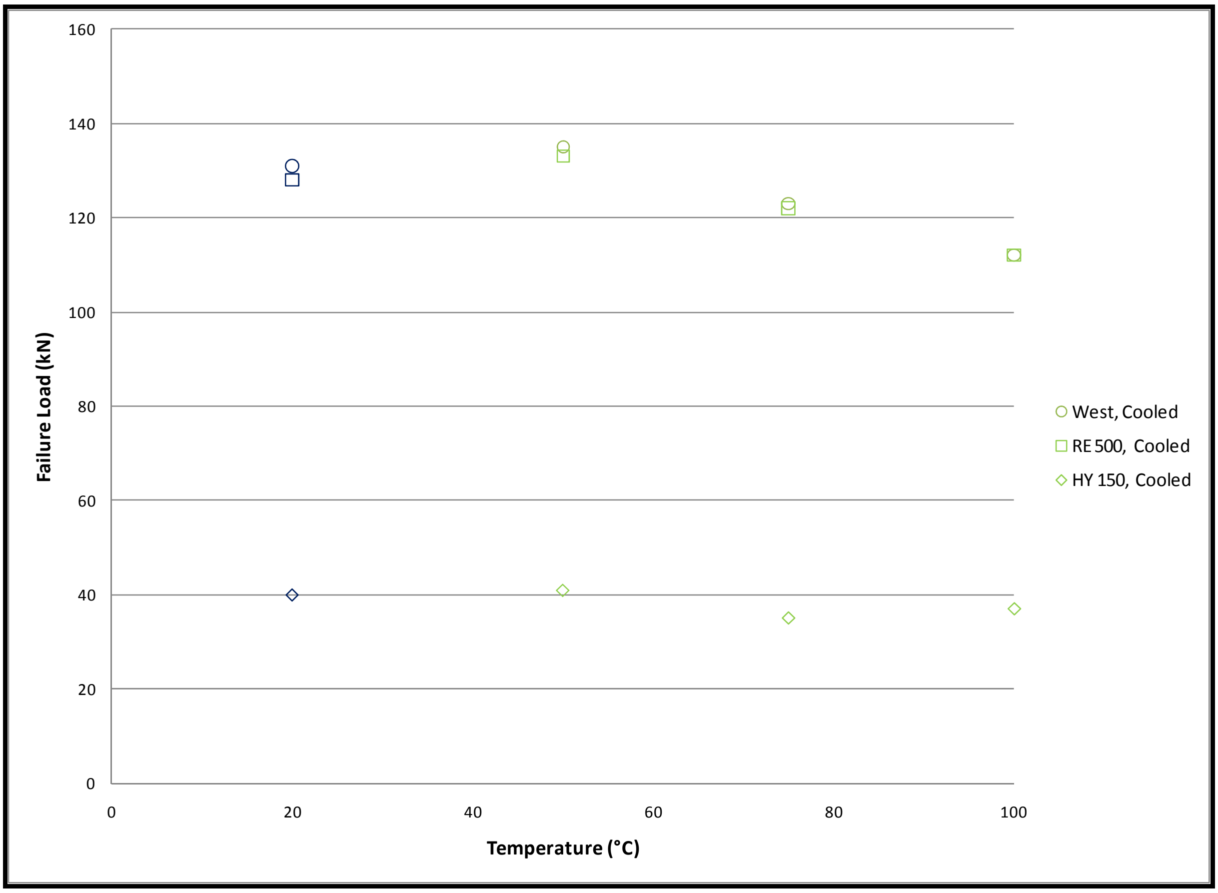

Figure 7-11 – Mean Cooled Test Data (Modified from Harris, 2004)

A comparison with experimental results and testing performed by Harris is shown in Figure 7-12. Test results are presented as a percentage of the ultimate strength at ambient temperature, to compare the relative strength loss for cooled specimens in both data sets.

Cooled test results indicate there is no significant strength difference between ultimate values for cold and cooled test specimens. The only significant variation in testing occurred with the Fischer high temperature epoxy, displaying the increase in cooled strength when compared to ultimate strength at ambient temperature conditions.

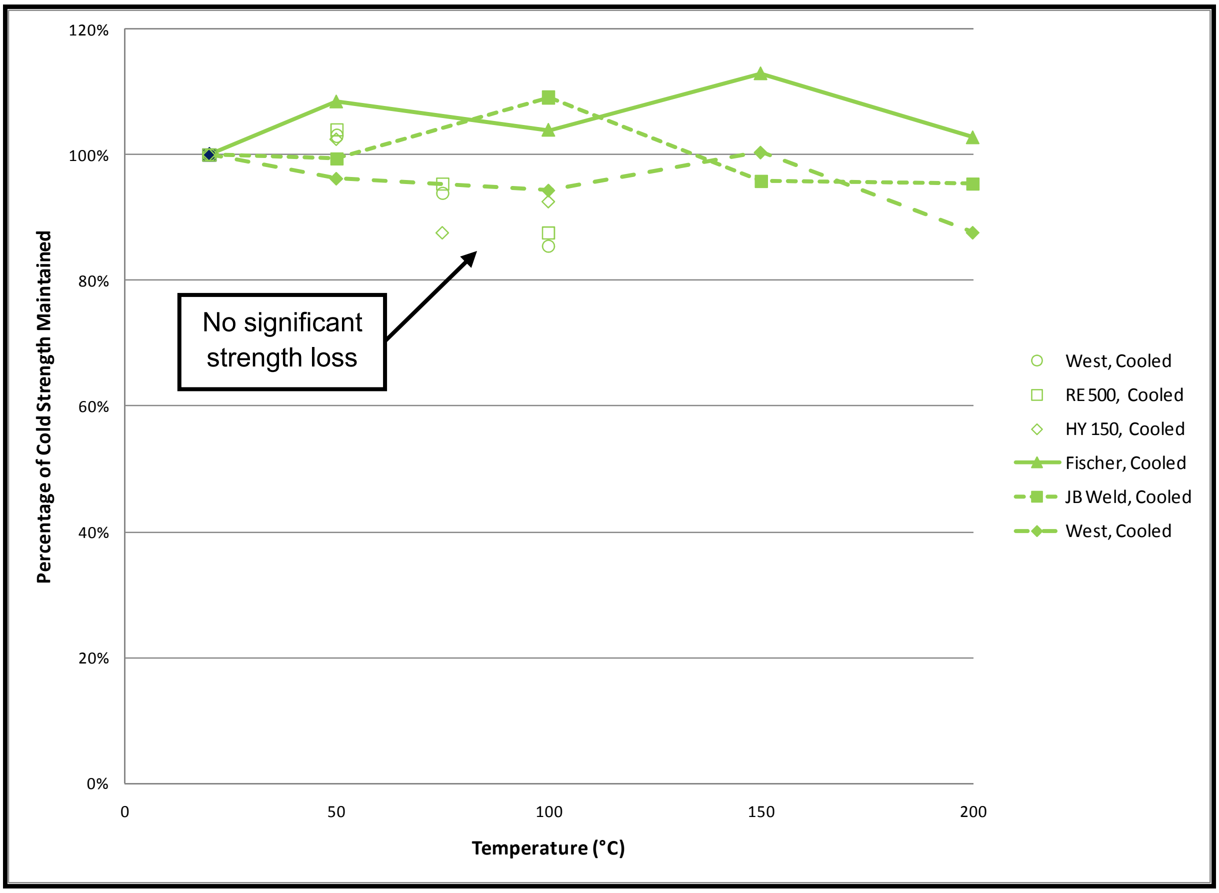

Figure 7-12 – Cooled Test Results - Harris (2004) Comparison

7.8. Conclusions

Cooled testing was performed to determine the behaviour and performance of high temperature epoxies subjected to heating and cooling. Epoxy grouted steel threaded rod specimens in LVL were heated overnight in the University of Canterbury Civil Engineering Laboratory oven to temperatures ranging from 50°C to 200°C, at 50°C increments. Specimens were allowed to cool to ambient temperature and subjected to tensile testing in the Avery Testing Machine. Cooled testing isolated the heating and cooling effect on high temperature epoxies, simulating a minor fire.

Comparing cooled test results with cold and oven test results as well as available literature provided insight into the behaviour and performance of epoxy grouted steel threaded rod connections. Comparisons with cold and oven data suggest that epoxy specimens demonstrated no significant strength loss when allowed to cool to ambient temperature. Tests specimens also displayed consistent failure mechanism behaviour and ultimate strength performance independent of heated temperature.

7.8.1. Epoxy Specimens

Previous cooled testing of epoxy grouted steel threaded rods suggested that epoxy subjected to heating and cooling experienced no significant change in ultimate strength (Harris, 2004). Experimental testing with Fischer, JB Weld and West high temperature epoxies confirmed this behaviour. Experimentation extended the temperature range beyond previously tested values and engaged in a comprehensive comparison of cold, oven and cooled test results.

Ultimate load values for cooled testing compared favourably with values obtained through cold testing. Surprisingly, the heating and cooling process increased ultimate strength values for the Fischer epoxy compared to cold test results. Further research could evaluate the strength increase resulting from the cooling procedure.

Comparing results from cooled tests with oven tests revealed significantly greater ultimate strength values for cooled test specimens. Cooled test results confirm that allowing the epoxy grouted test specimens to cool after a minor fire event allows the specimen to return to ultimate strength at ambient conditions.

Failure modes for cooled tests compared to cold and oven tests demonstrated consistency in behaviour and mechanism. When tested at ambient and lower temperatures, LVL confinement failures and epoxy grouted steel threaded rod pull out failures were the dominant failure methods. At high temperatures all epoxy samples displayed epoxy adhesive failures with pull out of the epoxy grouted steel threaded rod at the epoxy-wood interface.

Results from tensile tests using the three high temperature epoxy products evaluated the behaviour and performance of cooled epoxy grouted steel threaded rod connections. Testing confirmed that in a minor fire event, heating and cooling of epoxy does not significantly reduce the ultimate strength of the connection. Additional testing could evaluate experimental findings and provide more quantitative methods for establishing the performance of cooled steel to wood connections.

8. Furnace Testing

The final phase of experimental testing, furnace testing, was intended to determine the fire resistance for steel to wood connections. Epoxy grouted steel rod and proprietary test specimens were subjected to the ISO 834 standard fire in the custom furnace in the University of Canterbury Chemical and Process Engineering Laboratory. Results from furnace testing were used to calculate the fire resistance time for steel to wood connections.

8.1. Furnace Test Specimens

Furnace test specimens were identical to cold test specimens, except for one primary difference; specimens tested in the custom furnace were constructed with 650mm NelsonPine LVL to accommodate the furnace dimensions. Adjustments were made for two main reasons. First, specimen length was selected so the steel bracket would not be exposed to the heat within the furnace walls. Second, a longer specimen reduced the amount of insulation required to protect the exposed steel threaded rods.

8.1.1. Epoxy Specimens

All epoxy test specimens for furnace testing were constructed in the same process as for cold testing. This procedure can be found in Section 4.5.1. The only difference in furnace test specimens was the length increase to 650mm to accommodate the custom furnace dimensions. Epoxy grouted furnace test specimens are shown in Figure 8-1 and Figure 8-2.

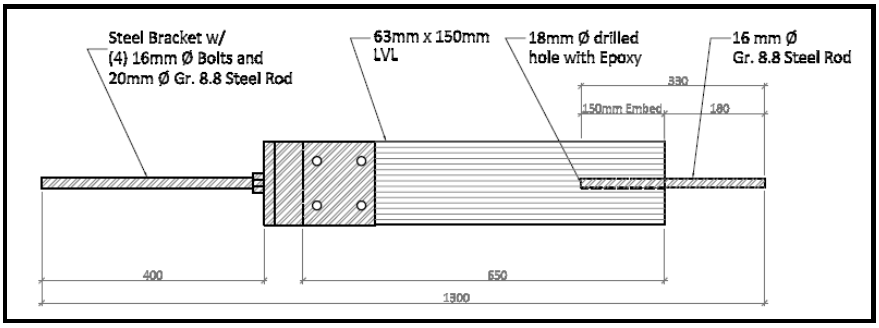

Figure 8-1 – Epoxy Furnace Test Specimen Schematic Drawing

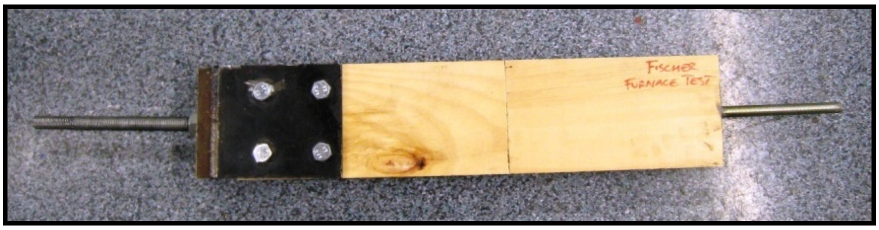

Figure 8-2 – Epoxy Furnace Test Specimen

8.1.2. Timberlinx Specimens

Timberlinx specimens were identical to cold, oven and cooled test specimens, with the only difference being the length increase to 650mm. Furnace test specimens were constructed following the procedure outlined in Section 4.5.2. Timberlinx furnace test specimens are shown in Figure 8-3 and Figure 8-4.

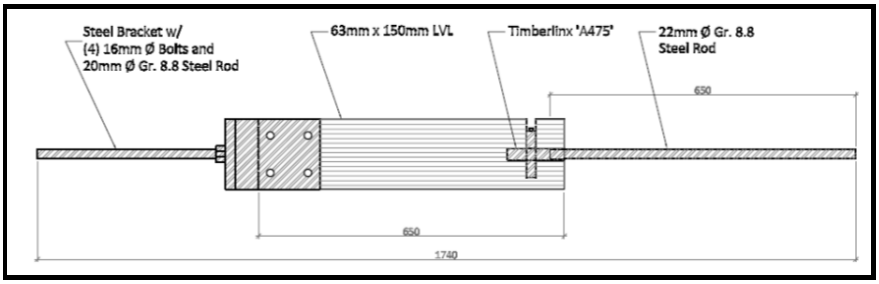

Figure 8-3 – Timberlinx Furnace Test Specimen Schematic Drawing

Figure 8-4 – Timberlinx Furnace Test Specimen

8.1.3. Lagscrewbolt Specimens

Lagscrewbolt test specimens used in the custom furnace were constructed to the same specifications as outlined in Section 4.5.3. The only difference in composition was the increase to 650mm. Lagscrewbolt furnace test specimens are shown in Figure 8-5 and Figure 8-6.

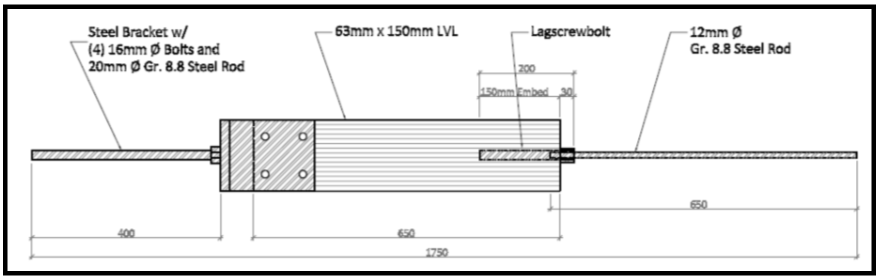

Figure 8-5 – Lagscrewbolt Furnace Test Specimen Schematic Drawing

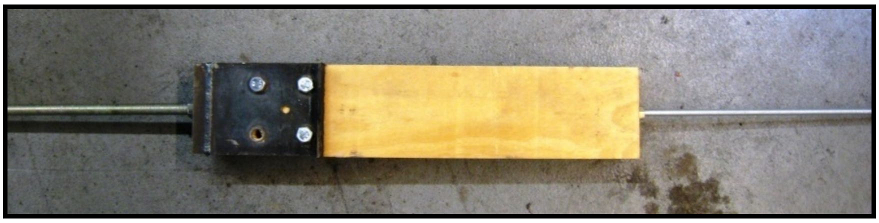

Figure 8-6 – Lagscrewbolt Furnace Test Specimen

8.2. Testing Procedure

Furnace testing was performed in the University of Canterbury Chemical and Process Engineering Laboratory. The three epoxy grouted steel threaded rod specimens and Timberlinx and Lagscrewbolt proprietary mechanical fasteners were loaded with a constant tensile force and subjected to the standard fire in the custom furnace. The amount of time the connection could sustain the constant load, equivalent to the failure time, was used to determine the fire resistance for the connection. Test specimens and results were used to evaluate the fire behaviour, performance, and resistance for the steel to wood connections.

8.2.1. Furnace Testing Preparation

Set up of the custom furnace involved the assembly of three main components for testing. The custom furnace was situated under the large hood air extraction device to remove smoke and ash from the laboratory. Once directly underneath the hood, the hydraulic ram was located on the table near the furnace. Both the furnace and the ram were hooked up to a laptop computer programmed with the Universal Data Logger (UDL) software to record experimental data.

Once all testing equipment was situated and was confirmed to work properly, specimens were loaded into the custom furnace in preparation for testing. Testing equipment including the custom furnace, hydraulic ram and laptop computer are shown in Figure 8-7:

Figure 8-7 – Custom Furnace Apparatus and Hood

8.2.2. Specimen Preparation

Steel to wood test specimens required additional preparation prior to furnace testing. Samples were fitted with the custom steel bracket and bolts were hand tightened to provide a firm connection at the restrained end of the specimen. Test specimens were then carefully centred within the custom furnace for equal heat application from the cylindrical furnace.

Efforts were made to prevent heat transfer through the LVL and to the exposed steel connections. Kaowool insulation was wrapped around the exposed portion of the steel threaded rod to prevent heat transfer through conduction throughout the specimen. Figure 8-8 displays the Kaowool insulation firmly pressed against the LVL and extending beyond the opening of the furnace.

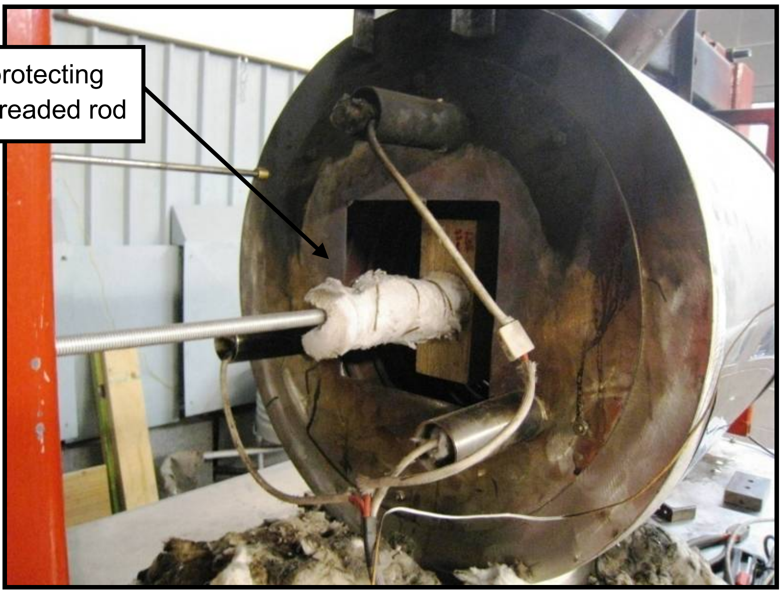

Figure 8-8 – Steel Rod Insulation

A fixed plate and nut at one side of the testing frame (at left in Figure 8-9) was tightened to secure the testing specimen in place. At the opposite end the steel threaded rod was secured to the hydraulic ram load-cell (at right in Figure 8-9) for tensile load application. Figure 8-9 displays the furnace test set up with the custom furnace, hydraulic ram, laptop computer and fully prepared test specimen ready to be subjected to furnace testing.

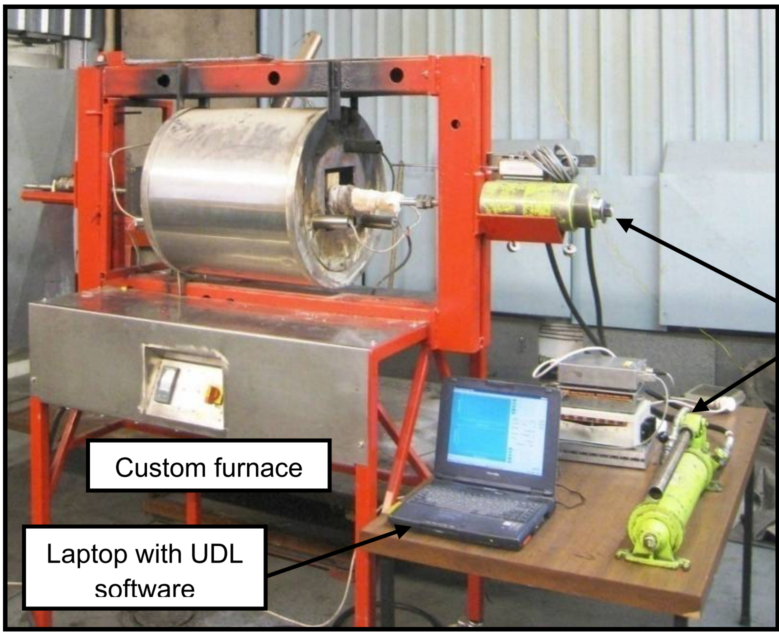

Figure 8-9 – Furnace Test Set Up

8.2.3. Load Application

Furnace testing involved the application of two types of loading: tensile load and a thermal load as specified by the ISO 834 standard fire (Buchanan, 2001). Tensile load was applied by the hydraulic ram attached to the test specimen. The temperature was specified by the fire load dial attached to the custom furnace.

Tensile testing in the custom furnace involved maintaining a constant tensile load on furnace test specimens. Typical design conditions require the consideration of multiple load combinations. The Universal Building Code requires load combinations of 1.2 times the dead load plus 1.6 times the live load for cold conditions and 1.0 times the dead load plus 0.4 times the live load for fire conditions (Buchanan, 2001).

To simplify the design process, a constant fire load was selected to replicate the load combination of dead load and permanent live load. The fire load was selected as 30% of the ultimate cold strength, in accordance with the constant tensile fire load selected in furnace tests performed by Harris (2004).

Table 8-1 presents the ultimate strengths obtained through cold testing and the constant fire load maintained during furnace testing:

Test Specimen	Ultimate Cold Strength (kN)	Fire Load (kN)
Fischer Epoxy	60.3	15
JB Weld Epoxy	66.8	20
West Epoxy	74.3	22
Timberlinx	36.0	11
Lagscrewbolt	42.7	13

Table 8-1 – Applied Tensile Loading

Tensile load was applied using a hydraulic ram attached to the steel threaded rod at the end of the furnace test specimen. Prior to testing, the test specimen was loaded to the prescribed tensile fire load. This constant fire load was maintained throughout the duration of the testing procedure. A slight leak in the hydraulic ram caused load to slowly decrease over time, but additional load was added as necessary to offset the loss. Minor fluctuations in constant load occurred during testing, however efforts were made to remain within a 0.5kN tolerance for applied tensile load. The hydraulic ram is shown at right in Figure 8-10:

Figure 8-10 – Hydraulic Ram (at right)

Full-scale fire tests utilise the ISO 834 standard fire curve for standardization of experimental results. The standard fire curve is a time-temperature curve dictating specified temperatures over a given amount of time. The ISO 834 standard fire curve, shown in Figure 8-11, is specified by the following equation (Buchanan, 2001):

$$T = 345\log_{10}(8t+1) + T_o \qquad \text{Equation 8-1}$$

$T = 345\log_{10}(8t+1) + T_o$

T = Temperature (°C)

t = Time (min)

T_o = Ambient temperature (°C)

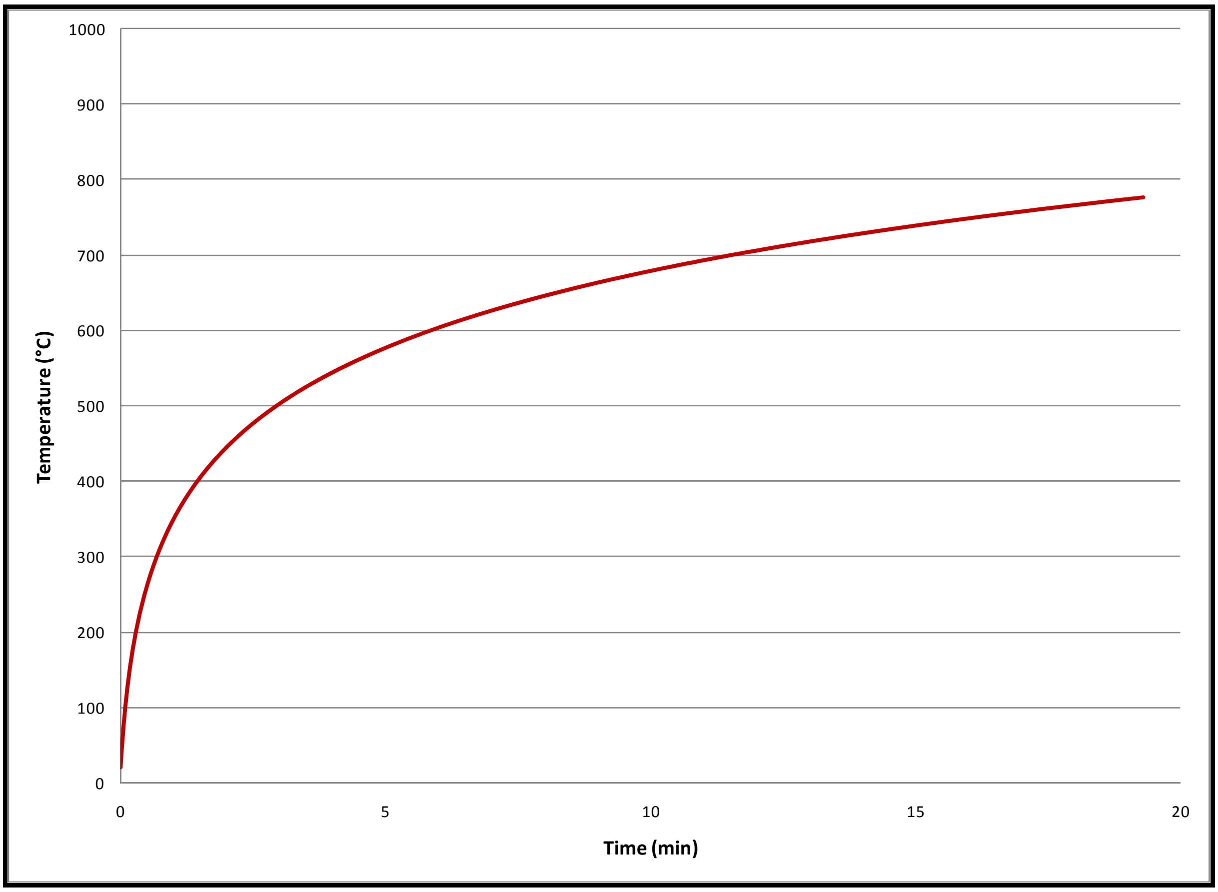

Figure 8-11 – ISO 834 Standard Fire Curve (Buchanan, 2001)

The fire temperature dial on the custom furnace was programmed to follow the standard fire curve throughout the duration of testing. Temperatures for the standard fire were programmed for a one hour duration. This provided more than enough time for experimental test specimen failure.

8.2.4. Data Recording

Output from experimental furnace testing was recorded using the Universal Data Logger software on the attached laptop computer. Both the applied load from the hydraulic ram and the furnace temperature were recorded. The UDL software recorded data from the beginning of load application prior to heating, through to specimen failure.

8.2.5. Specimen Failure

Connection failure was evaluated purely in the strength domain, independent of deformation. Failure was defined as the point at which the connection could no longer sustain the constant tensile load applied to the specimen.

Specimen failure for furnace tests, seen in Figure 8-12, often resulted in a loud popping sound consistent with a sudden, brittle failure mechanism. Once failure had occurred and the connection could no longer sustain the constant tensile load, the custom furnace was turned off and heating and recording were discontinued in preparation for post failure procedures.

Figure 8-12 – Furnace Test

8.2.6. Post Failure Procedure

Efforts were made to minimize the amount of post-failure charring to the specimens. Test specimens were quickly removed from the custom furnace (shown in Figure 8-13) and brought outside the laboratory to extinguish the flames and cool the specimens to prevent further charring.

Figure 8-13 – Test Specimen Removal

Once outside, specimens were placed on the ground and a fire hose was used to extinguish the existing flames. Water was sprayed until steam no longer rose from the test specimens (as seen in Figure 8-14). Specimens were rotated to provide equal cooling on all sides.

Figure 8-14 – Preventing Additional Charring

Test specimens displayed a significant amount of charring resulting from the furnace testing. After allowing the specimen to dry, the charring layer was removed to display the solid wood beneath. A hammer and wire brush (shown in Figure 8-15) were used to remove the charring layer that easily broke away from the undamaged wood.

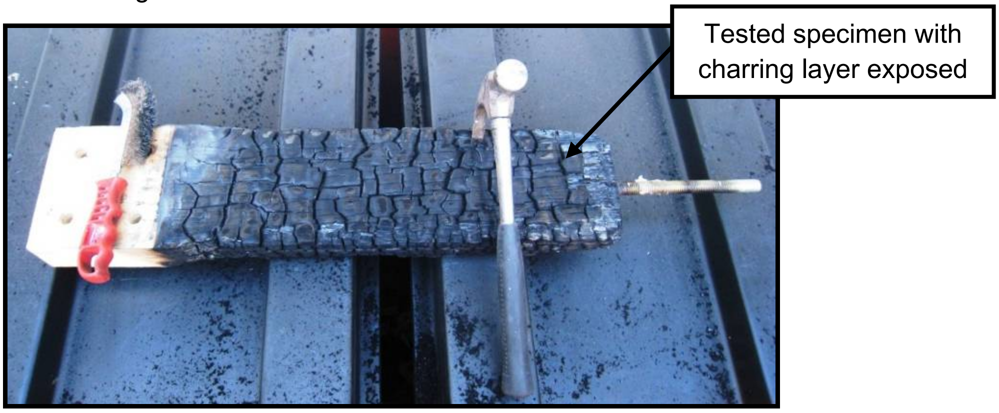

Figure 8-15 – Removal of Charring Layer

Once the charring layer was removed the wire brush was used to remove any additional charring and expose the undamaged portion of wood. This also exposed any cracking that occurred along the connection length. Removal of the charring layer allowed for closer inspection of furnace specimens, transforming test samples from the charred specimen seen in Figure 8-15 to the specimen seen in Figure 8-16:

Figure 8-16 – Final Charred Specimen

8.3. Results

Furnace testing was conducted with the custom furnace to determine the fire performance and fire resistance of steel to wood connections. High temperature epoxy grouted steel threaded rods and proprietary specimens were subjected to the ISO 834 standard fire under constant tensile loading to determine the length of time the specimen could sustain the loading prior to failure.

8.3.1. Fire Performance

Once specimens were secured in the custom furnace, the tensile load was applied and the specified magnitude confirmed by the UDL software. After tensile load confirmation, the furnace was turned on. This initiated the temperature increase following the ISO 834 standard fire protocol. As the temperature increased, the exposed LVL began to heat up, marked by charring and a build-up of pyrolyzates. As heating continued, a significant amount of smoke began to emanate from the furnace. After several minutes of heating, the LVL and gases reached the unpiloted ignition temperature and burst into flames.

Following ignition of the LVL specimen, significant flaming and charring was evident surrounding all sides of the testing samples. Based on observed flaming patterns, it was assumed that all sides of the experimental specimens experienced equal charring. Once failure occurred, the post failure procedure as presented in Section 8.2.6, was performed.

8.3.2. Time to Failure

Recording the time to failure during furnace tests was important for further use in establishing the fire resistance of each connection type. Recording began when the furnace was turned on and temperature began to increase within the furnace. The experimental duration was recorded by UDL software and continued through specimen failure; marked by the connection being unable to sustain any additional load. Time to failure values along with the constant furnace tensile load values are presented in Table 8-2:

Specimen	Furnace Load (kN)	Time to Failure (min)
Fischer	15	19.3
JB Weld	20	16.4
West	22	16.0
Timberlinx	11	15.7
Lagscrewbolt	13	14.4

Table 8-2 – Furnace Test Results

Time recording was stopped when the steel to wood connection failed to sustain the constant tensile load. UDL software recorded the applied load throughout the duration of testing. Specimen failure, marking the end of recorded time, can clearly be distinguished as the rapid drop in connection strength. The constant furnace tensile loads, as well as the time to failure values, are presented in Figure 8-17:

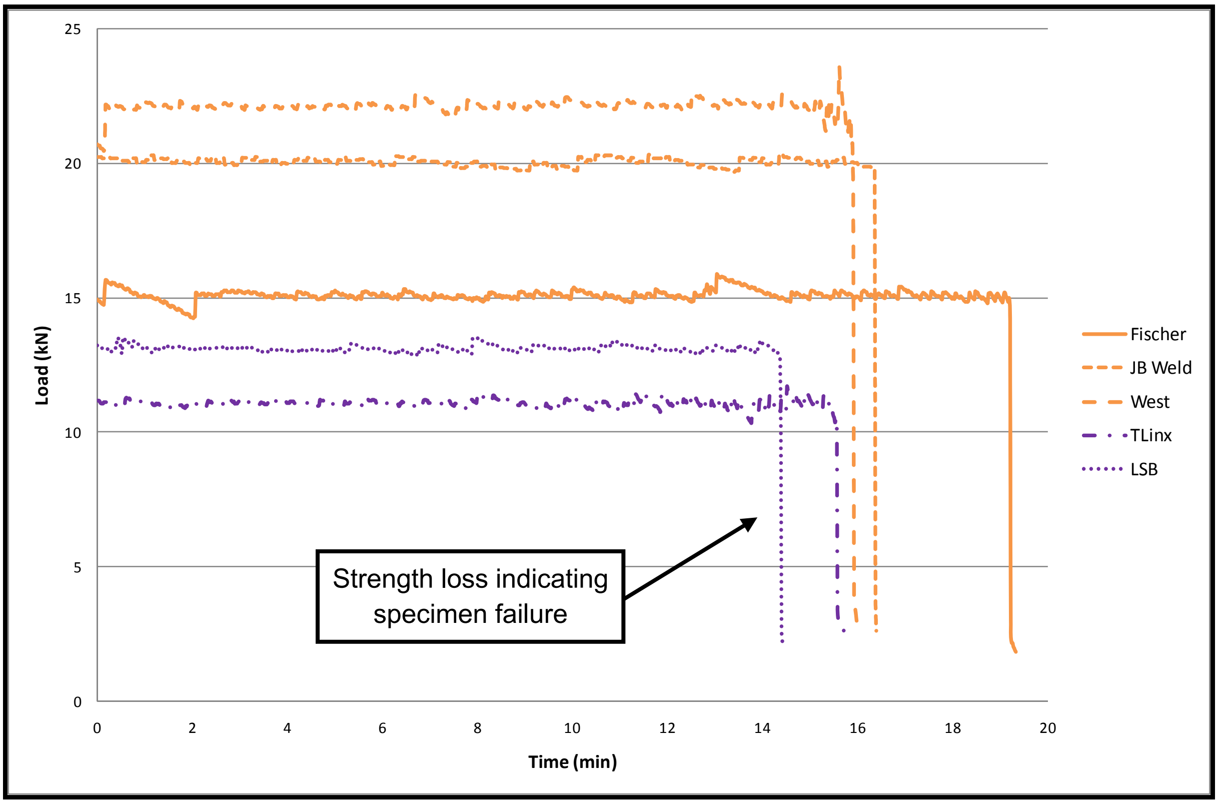

Figure 8-17 – Furnace Loads and Failure Times

8.3.3. Failure Modes

Furnace testing displayed two primary failure modes. The first failure mode, Failure Mode 1, occurred when the strength of the LVL was critically reduced by charring, resulting in a confinement failure in the wood and splitting perpendicular to the laminations at the critical edge distance. The Mode 6 Failure occurred only with the Timberlinx specimen and resulted in splitting of the LVL, representing a tension failure in the wood. Failure mode results are presented in Table 8-3:

Test Specimen	Failure Mode
Fischer	1
JB Weld	1
West	1
Timberlinx	6
Lagscrewbolt	1

Table 8-3 – Furnace Failure Modes

8.4. Failure Modes

Observing the failure modes that occurred during furnace testing in the custom furnace provided information regarding specimen behaviour and performance under fire conditions. Failure mechanisms were limited to brittle LVL failures.

8.4.1. Failure Mode 1

Mode 1 Failures were observed in nearly all furnace test specimens including all three types of epoxies and the Lagscrewbolt experimental specimen. Mode 1 Failures, as previously witnessed in cold and oven tests, are brittle confinement failures occurring in the LVL. The confinement failure is consistent with cracking perpendicular to the LVL laminations. Additional information on Failure Mode

1 can be found in Section 5.4.1. Failure Mode 1 can be seen in Figure 8-18 with a dark crack displayed along the length of the Lagscrewbolt connection:

Figure 8-18 – Failure Mode 1 in Lagscrewbolt Furnace Test Specimen

Observation of a number of Mode 1 Failures suggests several main conclusions. Primarily, the strength of the LVL was clearly the dominant failure mechanism for steel to wood connection samples. When charring occurred, the cross sectional area of the specimen exposed to the standard fire steadily decreased, reducing the overall strength of the wood. When the area decreased to a critical amount of wood cover along the smaller edge distance, it was no longer able to sustain the constant tensile force applied. This resulted in a confinement failure and cracking perpendicular to the wood at this reduced edge distance.

While being heated according to the standard fire, two primary events occurred. First, significant charring of the exposed LVL sections steadily decreased the cross sectional area. Second, conductive heat transfer occurred within the specimen, heating the embedded epoxy and Lagscrewbolt connections as the char layer continued to advance.

The heating process reduced strength in two ways; by reducing the cross sectional area of the LVL section and increasing the core temperature, thereby reducing strength in the epoxy connection. The failure mode analysis consistently demonstrated Mode 1 Failures, suggesting the strength of the LVL, and not the strength of the epoxy or steel specimens, was the dominant failure mechanism for steel to wood connections.

8.4.2. Failure Mode 6

The Mode 6 Failure occurred only in the Timberlinx specimen and resulted from a brittle LVL tensile failure parallel to the laminations. Specimen failure, as seen in Figure 8-19, occurred in line with the expanding anchor and nearly broke the experimental specimen into two distinct pieces, fracturing at one of the exposed sides.

Figure 8-19 – Failure Mode 6 in Timberlinx Furnace Test Specimen

The most striking feature of the Mode 6 Failure was its location. Drilling the hole for the expanding anchor bolt perpendicular to the steel threaded rod reduced the cross sectional area of the wood in the plane perpendicular to the laminations. When subjected to furnace testing, this already reduced cross sectional area was further reduced as the charring layer decreased the existing cross section. This decreased the LVL strength over time. When reaching a critical cross sectional area, the strength of the LVL could no longer sustain the constant tensile force and failure occurred with an abrupt cracking of the wood.

8.5. Fire Resistance Calculation

Fire resistance is a measure of how long a structural element can sustain a given design load when subjected to the standard fire. In terms of experimentation, the fire resistance represents the amount of time for which the given steel to wood connector can sustain the constant tensile loading when subjected to the ISO 834 standard fire. Results from furnace testing were used to calculate the fire resistance of steel to wood connections using the equation established by Harris (2004) to account for differences between the custom furnace and ISO 834 furnace.

8.5.1. Charring Rate

Analyzing the charring rate was necessary for calculating the fire resistance of steel to wood connections. The average charring rate for furnace test specimens was calculated using pre and post fire specimen thicknesses and the time to failure.

Following testing the charring layer was removed from furnace test specimens to expose the residual wood cross section. This residual thickness was measured and the charring depth was calculated by subtracting the residual thickness from the initial thickness and dividing by two for both sides of the specimen exposed to fire. The average charring rate was calculated by dividing the charring depth by the time to failure. Furnace test results and the average charring rate analysis are presented in Table 8-4:

Specimen	Thickness (mm)	Residual Thickness (mm)	Charring Depth (mm)	Time to Failure (min)	Average Charring Rate (mm/min)
Fischer	63	31	16	19.3	0.829
JB Weld	63	35	14	16.4	0.854
West	63	36	13.5	16.0	0.844
Timberlinx	63	36	13.5	15.7	0.860
Lagscrewbolt	63	38	12.5	14.4	0.868

Table 8-4 – Furnace Test Results with Char Rate

This average charring rate represents the average amount of charring occurring per minute of furnace testing. Calculating the average for the charring rate accounts for the differences in the charring rate between the initial minutes of testing prior to ignition and through to failure.

Attempts were made to simulate the standard fire curve as best as possible in the custom furnace. Differences in temperature, however, caused variation in the average charring rate for furnace tests compared to the accepted charring rate for LVL. Full-scale furnace tests using LVL subjected to the ISO 834 standard fire resulted in a charring rate of 0.72mm/min for LVL (Lane, 2001). Differences between the average charring rate in the custom furnace and the accepted charring rate for LVL found by Lane are mitigated through the use of a conversion equation presented by Harris (2004) in Section 8.5.2.

8.5.2. Fire Resistance Analysis

Harris (2004) presented an equation that introduced a scale factor to account for differences in the charring rate. The time to failure, found through custom furnace testing, was converted to a fire resistance time, found by a furnace prescribed to the ISO 834 standard fire, using the equation presented in Equation 8-2:

$$T_{ISO} = \frac{c_{cust}}{c_{ISO}}\left(t_{cust}\right)$$ **Equation 8-2**

T_{ISO} = Equivalent fire resistance time in ISO 834 Furnace (min)

c_{cust} = Charring rate for custom furnace (mm/min)

c_{ISO} = Charring rate for ISO 834 Furnace (mm/min)

t_{cust} = Time to failure in custom furnace (min)

The equivalent fire resistance for steel to wood connections was calculated by combining the average charring rate and time to failure found in custom furnace testing, with the accepted charring rate of 0.72mm/min, as found through ISO 834 furnace testing (Lane, 2001). Results for the fire resistance calculation are shown in Table 8-5:

Specimen	Thickness (mm)	Residual Thickness (mm)	Charring Depth (mm)	Time to Failure (min)	Charring Rate (mm/min)	Fire Resistance (min)
Fischer	63	31	16	19.3	0.829	22.5
JB Weld	63	35	14	16.4	0.854	19.7
West	63	36	13.5	16.0	0.844	19.0
Timberlinx	63	36	13.5	15.7	0.860	19.0
Lagscrewbolt	63	38	12.5	14.4	0.868	17.6

Table 8-5 – Furnace Test Results with Fire Resistance

A fire resistance analysis suggests that the Fischer epoxy displayed the greatest fire resistance for steel to wood connections. The Fischer high temperature epoxy sample maintained a constant tensile load for an equivalent fire resistance time of 22.5 minutes. The other two high temperature epoxies, as well as the Timberlinx connector, displayed fire resistance in the 19-20 minute range. The Lagscrewbolt proprietary steel to wood connector displayed the lowest fire resistance at just less than 18 minutes.

A comparison of failure time versus fire resistance is shown in Figure 8-20. The trend line correlating the two axes represents a scale factor for custom furnace charring rate divided by the accepted charring rate for LVL as provided by Lane (2001).

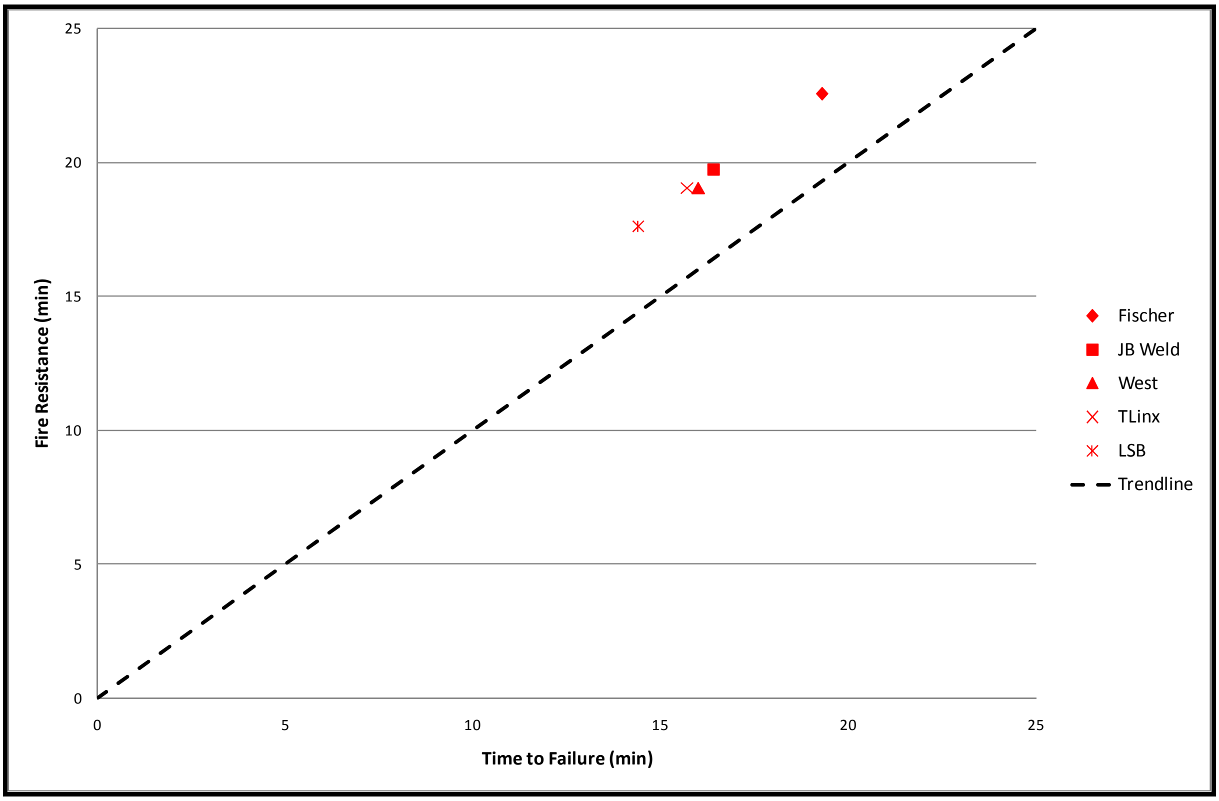

Figure 8-20 – Time to Failure - Fire Resistance Comparison

Data points indicate that the fire resistance is actually greater than the time to failure. This means that the scale factor was greater than 1.0 and the charring rate in the custom furnace was greater than the accepted charring rate. In effect, the custom furnace fire was actually more severe than the ISO 834 standard fire. An analysis for temperature and energy comparison between the custom furnace and standard fire is shown in Section 8.7.

8.6. Comparison with Previous Testing

Comparing results from furnace testing with previous furnace test results provides a validation process by which current testing can be evaluated. Previous furnace testing by Lane (2001) and Harris (2004) was performed to determine the fire behaviour and performance of laminated veneer lumber and epoxy grouted steel threaded rods. Furnace test results from current experimentation were compared with available data to evaluate the fire resistance for steel to wood connections.

8.6.1. Lane (2001)

Previous testing by Lane (2001) investigated the fire performance of laminated veneer lumber through the use of comprehensive ignition and charring tests. Lane performed extensive fire testing of Radiata Pine LVL samples in a full-scale pilot furnace implementing the ISO 834 standard fire. Furnace testing resulted in a cumulative charring rate of 0.72mm/min for Radiata Pine LVL (Lane, 2001).

Lane's charring rate for LVL has been used as a baseline charring rate for determining an equivalent fire resistance for steel to wood connections. A charring rate comparison for the average charring rate for experimental specimens and the accepted charring rate for Radiata Pine LVL is shown in Table 8-6:

Average Charring Rate	Accepted Charring Rate (Lane, 2001)	Percent Difference
0.851 mm/min	0.72 mm/min	18.2%

Table 8-6 – Charring Rate - Lane (2001) Comparison

Observing the charring rate for current experimentation and the accepted charring rate for LVL reveals a considerable percent difference of 18.2%. This is due to the differences in fire severity for the custom furnace and the ISO 834 furnace used by Lane. In addition, NelsonPine LVL uses a combination of Radiata Pine and Douglas Fir, which could account for slight variation in wood charring rates. Ultimately, however, the use of the scale factor in Equation 8-2 accounts for charring rate discrepancies between the custom furnace and ISO furnace.

8.6.2. Harris (2004)

Testing performed by Harris (2004) investigated the fire resistance of epoxy grouted steel rods in laminated veneer lumber. Furnace tests using 63mm x 63mm Radiata Pine LVL and all-purpose epoxy adhesive provide a source of information for comparison with current furnace testing. Charring rate values for epoxy grouted LVL in current testing and experimentation performed by Harris are shown in Table 8-7:

Average Epoxy Specimen Charring Rate	Average Charring Rate (Harris, 2004)	Percent Difference
0.842 mm/min	0.47 mm/min	44.2%

Table 8-7 – Charring Rate - Harris (2004) Comparison

The average epoxy specimen charring rate of 0.842mm/min appears considerably greater than the 0.47mm/min found by Harris. The difference in values is likely due to the major improvements to the custom furnace between testing. Harris noted that furnace testing had to be discontinued due to malfunctioning of the electrical heat flux coils within the custom furnace. Re-commissioning of the furnace remedied this issue by replacing and upgrading the electrical heat flux coils. This provided greater precision in replicating the ISO 834 standard fire curve in future furnace testing.

Several factors are likely to affect the difference in fire resistance between the testing samples. These include:

- Differences in the custom furnace charring rate and the 0.72mm/min charring rate for LVL (Lane, 2001).
- Different LVL specimen sizes that act as a cover depth for the connection (63x63mm and 63x150mm). Note: the governing depth (63mm) was consistent for all testing.
- The use of all-purpose epoxy in previous testing and high temperature epoxy in the current testing regimen.

A failure time and fire resistance comparison for experimental furnace testing and Harris' results are shown in Table 8-8. The average failure time for each specimen is the total sum of the failure times divided by the number of failures for each specimen type. The average fire resistance is equal to the average failure time with the included scale factor accounting for differences in fire severity. A plot displaying the furnace testing results is shown in Figure 8-21.

Specimen	Average Failure Time (min)	Average Fire Resistance (min)
Fischer	19.3	22.5
JB Weld	16.4	19.7

West	16.0	19.0
West (Harris, 2004)	15.3	8.7
RE 500 (Harris, 2004)	15.5	9.7
HY 150 (Harris, 2004)	23.3	17.3

Table 8-8 – Epoxy Furnace Test Results - Harris (2004) Comparison

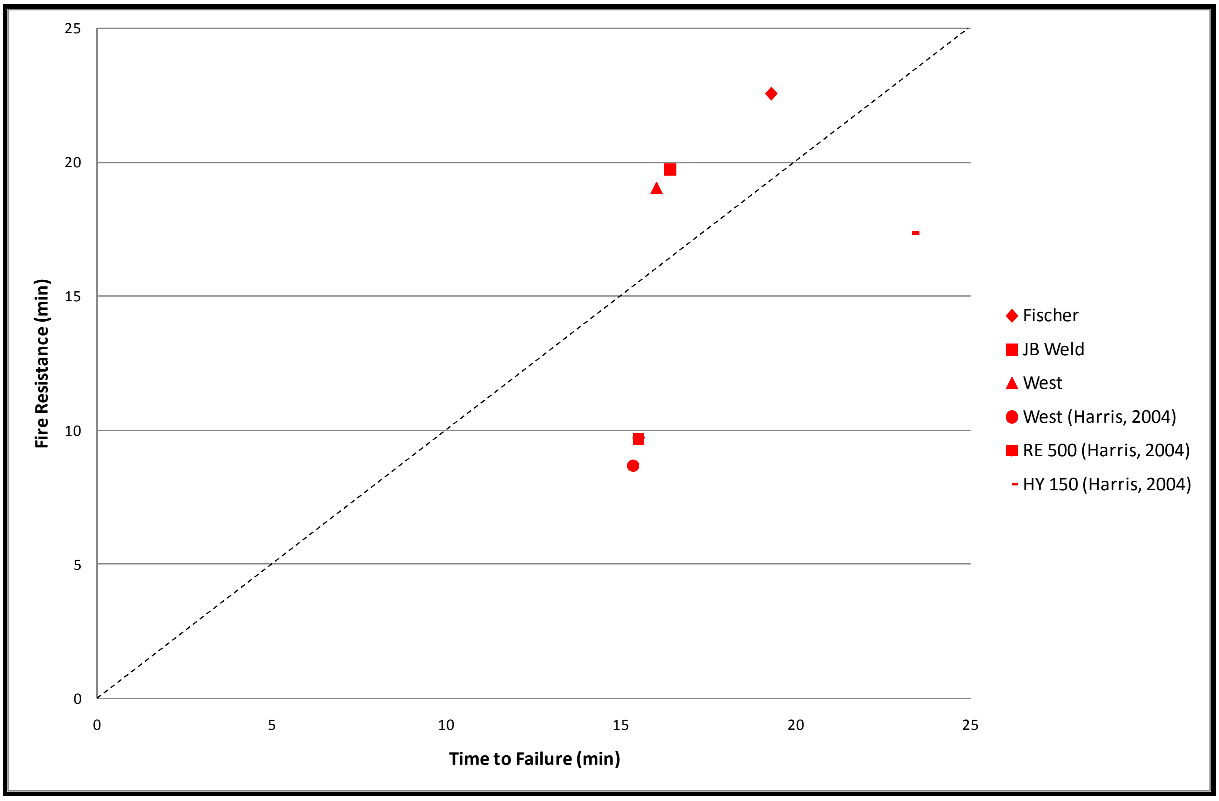

Figure 8-21 – Epoxy Furnace Test Results – Harris (2004) Comparison

Observing the furnace test results indicates that while the average times to failure were comparable, the fire resistance for the experimental high temperature epoxies significantly outperformed the all-purpose epoxy adhesives. The significant difference in charring rate greatly affected the scale factor when converting to a standard fire resistance. This difference in scale factor caused the fire resistance for Harris' samples to decrease less than the time to failure, while high temperature epoxy specimens increased and demonstrated more favourable fire resistance times.

Differences in values could result from several possibilities. The most obvious is the size of the LVL specimen. Furnace test specimens in Harris' experimentation were 63mm square LVL, while current research used 63mm x 150mm LVL sections.

Despite the difference in specimen sizes, the primary difference was the type of epoxy used in testing. Previous all-purpose epoxies were shown to behave poorly under fire conditions (Barber, 1994 and Harris, 2004). Current research using high temperature epoxies displayed significantly improved performance. Fire resistance times from specimens using high temperature epoxies were more than double previous tested values. This suggests that high temperature epoxies display considerably improved fire performance and fire resistance than previously tested specimens.

Current furnace testing results have witnessed LVL failures occurring in all samples, generally due to confinement failures at the critical edge. Testing performed by Harris, however, observed a number of epoxy failures for furnace test specimens. As the temperature of the epoxy increased, the adhesive strength decreased, causing an epoxy failure at the epoxy-wood interface. Witnessing an epoxy failure as opposed to an LVL failure suggests that the strength of the epoxy decreased at a faster rate than the strength of the wood. An example of an epoxy failure for a furnace test specimen can be seen in Figure 8-22:

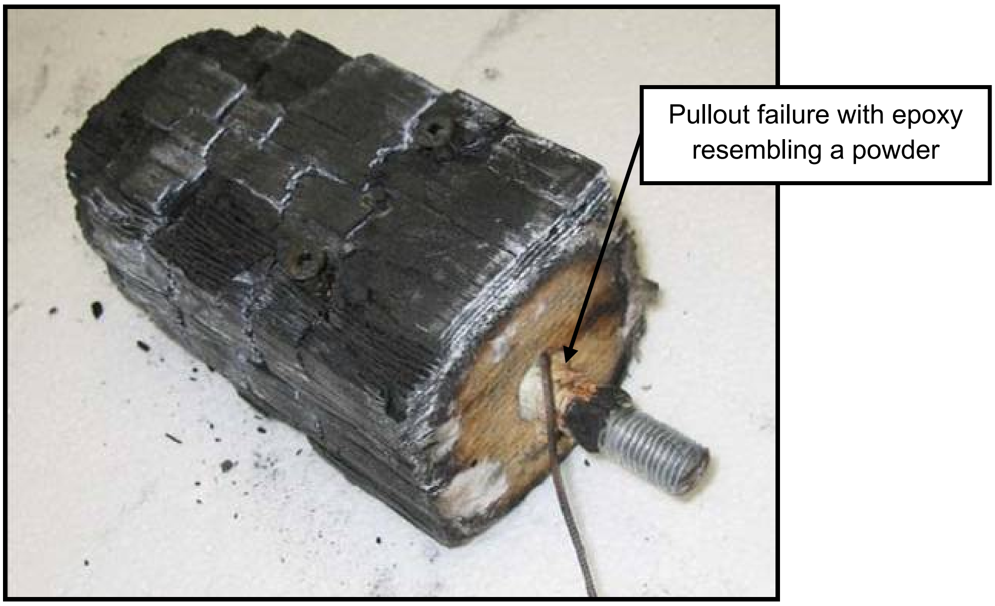

Figure 8-22 – Epoxy Failure for Furnace Test Specimen (Harris, 2004)

While previous testing has demonstrated that all-purpose epoxy loses strength at high temperatures, furnace testing did not result in a single epoxy failure. This suggests that the strength of the LVL cover was the governing failure mechanism, as there was no evidence of high temperature epoxy failure in furnace testing. Compared to previous results with all-purpose epoxy, results with high temperature epoxy suggest improved fire performance.

8.7. Comparison with the ISO 834 standard fire

The ISO 834 standard fire is universally used to standardize full-scale fire tests. While this prescribed standard fire curve prescribes a given time and temperature throughout the testing procedure, differences in types of furnaces and furnace conditions may cause variation in results (Buchanan, 2001). Comparing custom furnace temperatures with temperatures prescribed by the ISO 834 standard fire curve allows for a temperature evaluation for justifying furnace test results.

8.7.1. Temperature

Thermocouples placed within the centre of the custom furnace flue and connected to the UDL data recording software recorded the furnace temperature throughout the experimentation process. Testing began at ambient temperature and followed the standard fire curve as closely as possible. Temperatures for each furnace test were averaged over the full duration of testing. The resulting average furnace temperature is presented with the standard fire curve in Figure 8-23:

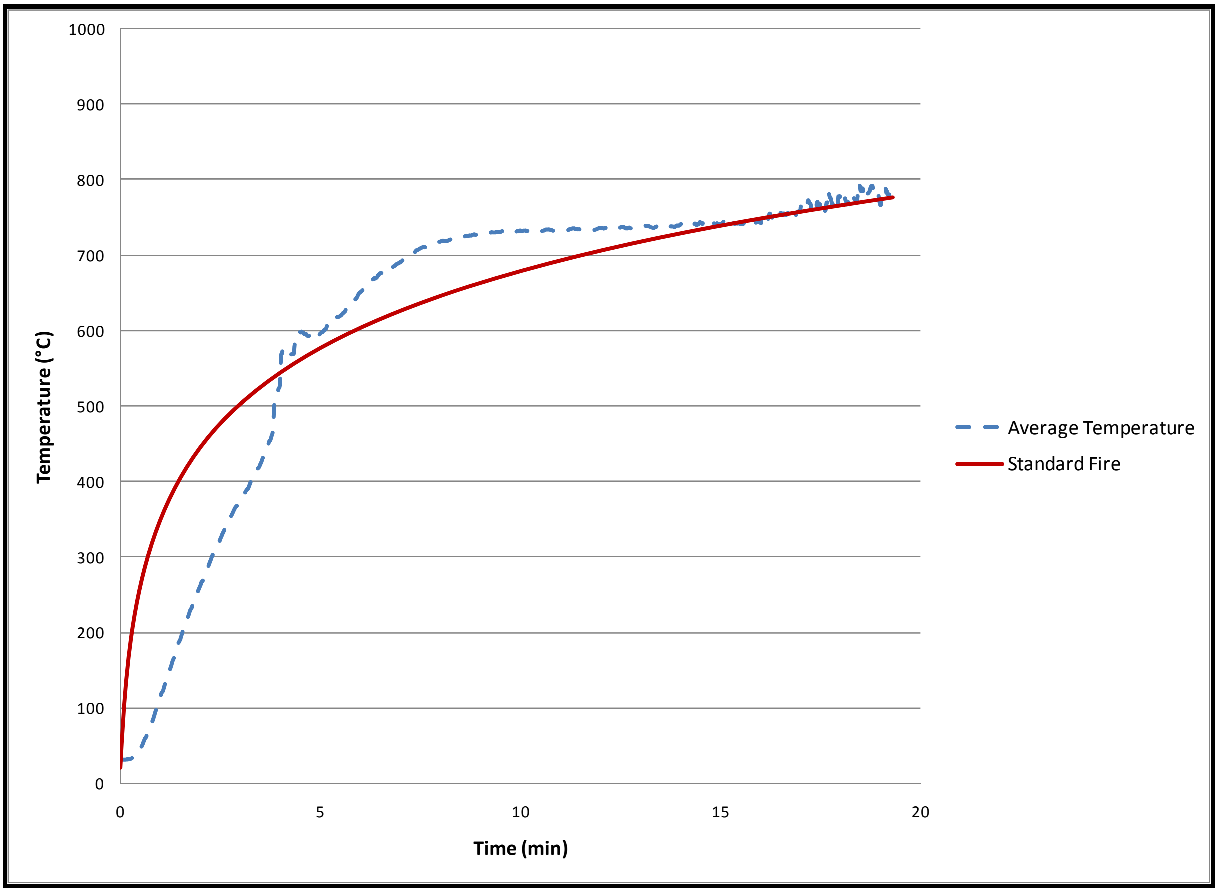

Figure 8-23 – Average Temperature - Standard Fire Temperature Comparison

The furnace temperature generally follows the prescribed temperature established by the ISO 834 standard fire curve. The furnace temperature heats slower than the curve in the first few minutes and over-compensates prior to levelling out around the 15 minute mark.

8.7.2. Energy

To account for discrepancies in fire severity, the cumulative radiant energy, representing the area of a plot of radiant energy versus time, can be used to make a comparison between fire scenarios (FEDG, 2008). The average temperature from the custom furnace tests and the temperature from the standard fire were converted into cumulative radiant energy values for comparison using Equation 8-3:

$$E = \int_0^t \dot{Q}'' dt = \varepsilon\sigma\int_0^t \left(T^4\right)dt$$ **Equation 8-3**

E = Cumulative radiant energy over a period of time (J/m^2)

t = Time from the start of the test (sec)

$\dot{Q}''$ = Radiant heat flux at any point in time (W/m^2)

ε = Emissivity (assume 1.0)

σ = Stefan Boltzman constant (5.67*10^{-8} W/m^{2}K^4)

T = Temperature (K)

After converting the temperature from Celsius to Kelvin, Equation 8-3 was used for each one second interval throughout the full testing duration. A plot of the radiant energy for the custom furnace and standard fire is shown in Figure 8-24:

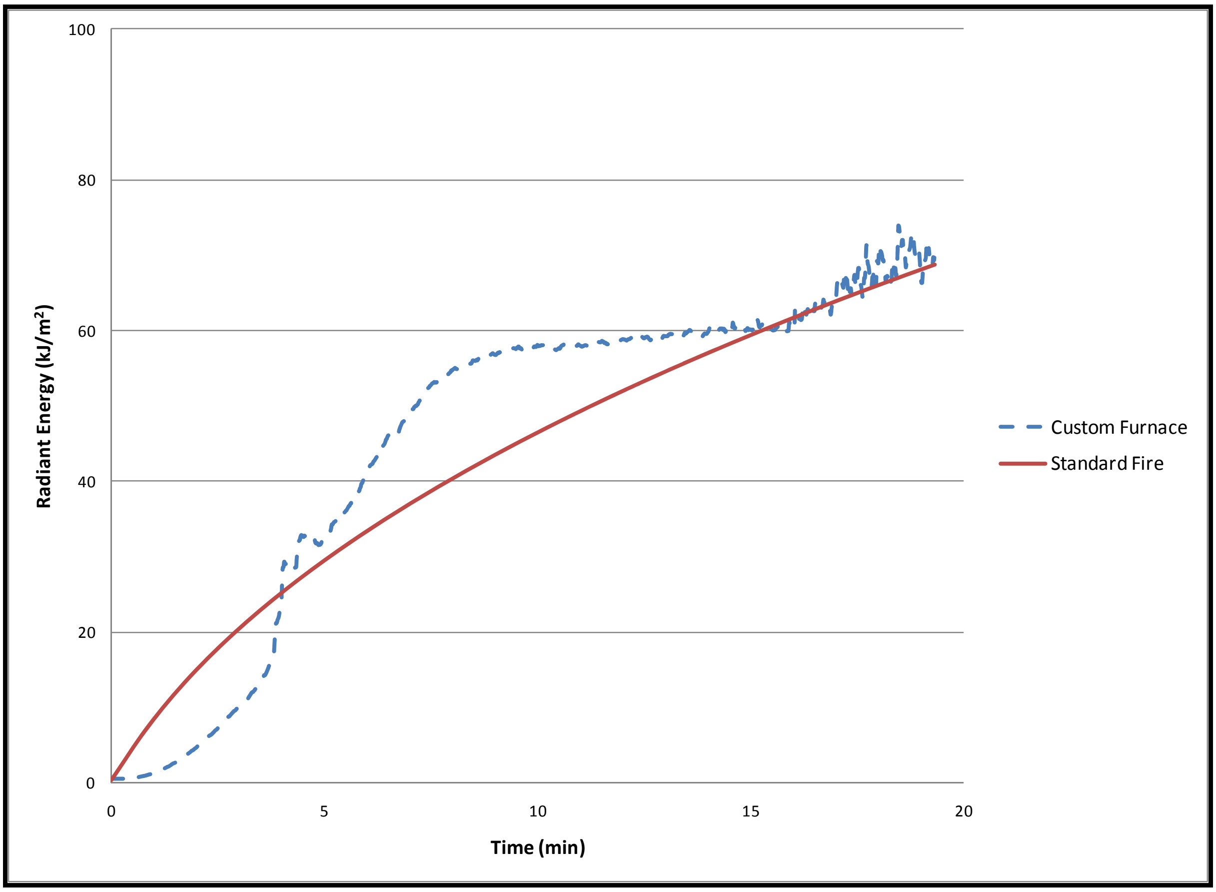

Figure 8-24 – Average Furnace - Standard Fire Radiant Energy Comparison

The radiant energy comparison appears similar to the temperature comparison for the custom furnace and standard fire curves. The furnace radiates less energy than the standard fire up to five minutes, then over-compensates until levelling and displaying comparable behaviour beyond 15 minutes.

Taking the sum of each one second interval for both time-energy curves gives the cumulative radiant energy for the custom furnace and the standard fire. These values can be seen with a percent difference calculation presented in Table 8-9:

Average Furnace Energy	Standard Fire Energy	% Difference
$5.06*10^4$ kJ/m^2	$4.71*10^4$ kJ/m^2	6.88%

Table 8-9 – Average Furnace - Standard Fire Cumulative Radiant Energy Comparison

A comparison between the average furnace and ISO 834 standard fire cumulative radiant energy displays a 7% difference in value. This demonstrates that the custom furnace is more severe than the standard fire by approximately 7%. This difference in fire severity helps to explain the custom furnace charring value being greater than the previously accepted charring value established by Lane (2001).

8.8. Fire Resistance Calculation

Results from furnace testing can be used to establish equations that not only analyze, but also design for the fire resistance of steel to wood connections. Equations are based on data from experimental furnace testing, and assume a charring rate of 0.72mm/min for LVL subjected to the standard fire (Lane, 2001).

8.8.1. Fire Resistance Design

When designing structural connections it is important to be aware of the fire resistance for structural elements and connections to provide fire safety. Equation 8-4 provides a quantitative method for calculating the cover depth required to protect steel to wood connections to specify a desired fire resistance:

$$d_{cover} = (t_{FR})(c_{ISO}) + \frac{(b_{fail} - \varphi_{hole})}{2} \qquad \textbf{Equation 8-4}$$

d_{cover} = Required cover depth to attain desired fire resistance (mm)

t_{FR} = Fire resistance time desired (min)

c_{ISO} = Charring rate for LVL in the ISO 834 Furnace (mm/min)

b_{fail} = Failure width for steel to wood connection (mm)

φ_{hole} = Hole diameter for steel to wood connection (mm)

Equation 8-4 can be used in combination with furnace results (provided in Table 8-10) to calculate the minimum amount of cover necessary for meeting a specific fire resistance time. Figure 8-25 presents a visual display of the equation variables:

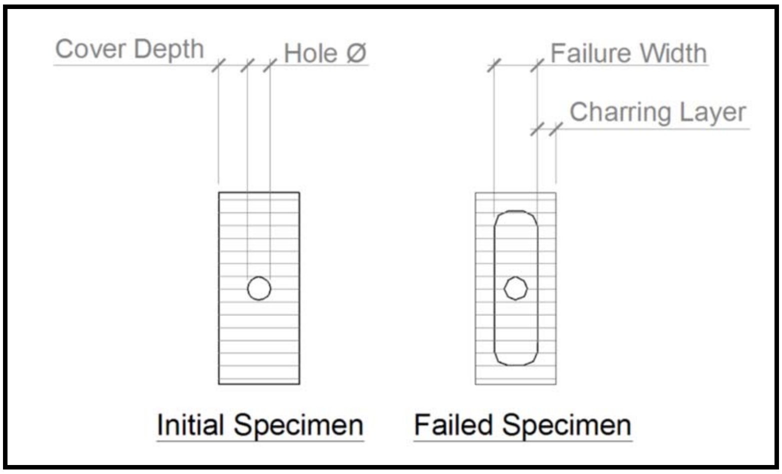

Figure 8-25 – Pre and Post Failure LVL Specimens

Specimen	Failure Width (mm)	Hole Diameter (mm)
High Temperature Epoxy	34mm	18mm
Timberlinx ‘A475’	36mm	24mm
Lagscrewbolt	38mm	22mm

Table 8-10 – Experimental Furnace Results

For example, Equation 8-4 can be used to determine the minimum required cover depth to provide a 30 minute fire resistance for a steel to wood connection using high temperature epoxy:

$$d_{cover} = (30\min)(0.72mm/\min) + \frac{(34mm - 18mm)}{2}$$

$$d_{cover} = 29.6mm$$

Inserting the 30 minute specified fire resistance, the 0.72mm/min charring rate for LVL and taking values for failure width and hole diameter for high temperature epoxy from Table 8-10, the minimum required cover depth is equal to 29.6mm. Users should be aware that this is the minimum calculated cover and a margin of safety should be included for conservative design.

Table 8-11 and Figure 8-26, shown below, use the fire resistance design equation in Equation 8-4 to determine the required cover depth. In Table 8-11, one must first select the desired fire resistance, ranging from 10 to 60 minutes, and continue across the row to find the required cover depth (in mm) for each of the three experimental steel to wood connections.

Fire Resistance (min)	High Temperature Epoxy	Timberlinx	Lagscrewbolt
10	15	13	15
20	22	20	22
30	30	28	29
40	37	35	37
50	44	42	44
60	51	49	51

Table 8-11 – Minimum Required Cover for Fire Resistance (mm)

The chart in Figure 8-26 provides an identical method for determining the required cover. The first step is to select a desired fire resistance on the x-axis, then continue upwards until hitting the curve for which the connection is to be designed. Going across to the x-axis provides the required cover depth to accomplish the design fire resistance.

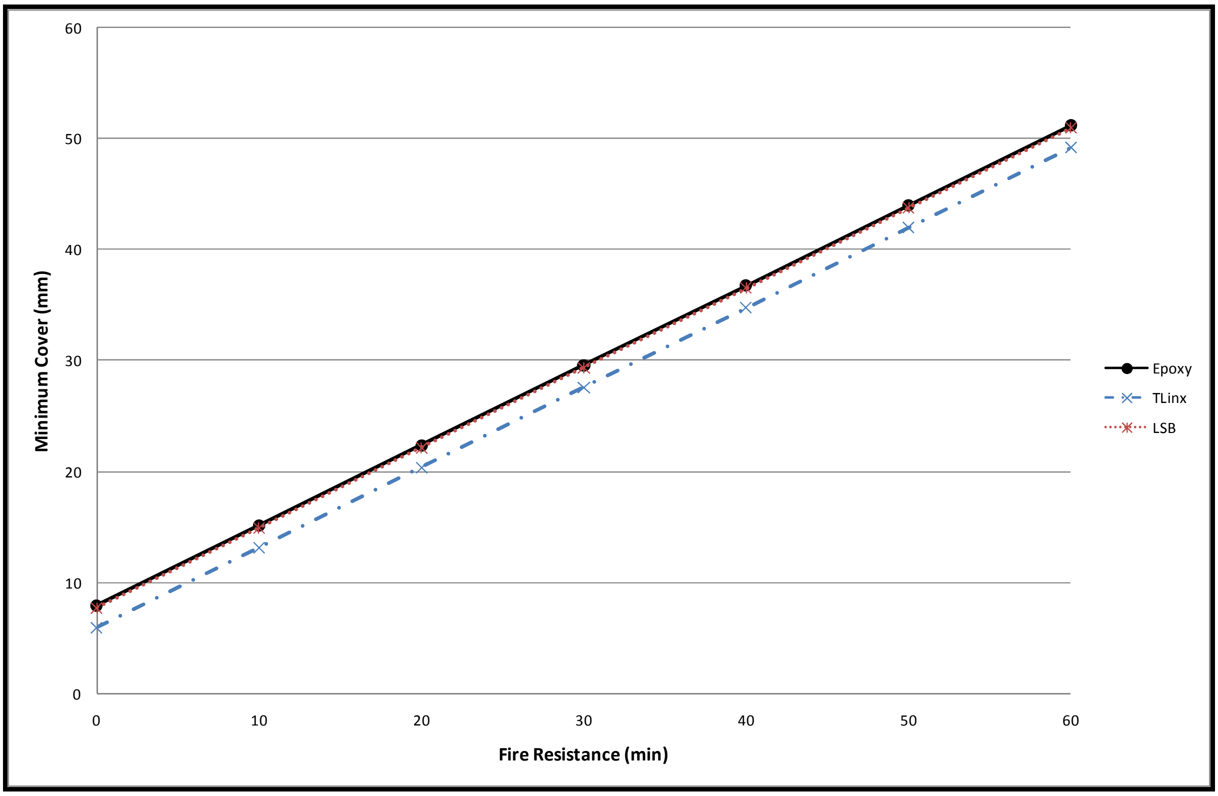

Figure 8-26 – Fire Resistance Design Plot

8.8.2. Fire Resistance Analysis

Furnace test results can also be used to determine the fire resistance for existing steel to wood connections. Equation 8-5, shown below, combines the existing cover depth with furnace test results and the accepted charring rate for LVL in the ISO 834 Furnace to determine the fire resistance of existing steel to wood connections using the following:

$$t_{FR} = \frac{d_{cover} - \left(\frac{b_{fail} - \varphi_{hole}}{2}\right)}{c_{ISO}} \qquad \textbf{Equation 8-5}$$

t_{FR} = Fire resistance time for given depth of cover (min)

d_{cover} = Cover depth (mm)

b_{fail} = Failure width for test specimens (mm)

φ_{hole} = Hole diameter for steel to wood connection (mm)

c_{ISO} = Charring rate for LVL in the ISO 834 Furnace (mm/min)

To calculate the fire resistance time for a given section, furnace test results (provided in Table 8-10) are combined with the 0.72mm/min charring rate in Equation 8-5. A sample calculation to find the fire resistance time for a Timberlinx specimen with 50mm of LVL cover, a 36mm failure width and a 24mm hole diameter is presented as follows:

$$t_{FR} = \frac{50mm - \left(\frac{36mm - 24mm}{2}\right)}{0.72mm/\min}$$

$$t_{FR} = 61.1\min$$

Inserting the corresponding variables from Table 8-10 into Equation 8-5 yields a fire resistance time of 61.1min. Applying Equation 8-5 and rounding down to the nearest 30 minute increment suggests a Timberlinx specimen with 50mm of LVL cover can be expected to have a 60 minute fire resistance.

Equation 8-5 calculates the fire resistance time for a specified cover depth for steel to wood connections in LVL. This equation can be expanded to all three connections, the high temperature epoxy, Timberlinx 'A475' and Lagscrewbolt products, to determine the fire resistance time for a range of cover depths. Table 8-12 presents the fire resistance times for all three types of steel to wood connections for cover depths ranging from 10mm to 100mm. One must first select the cover depth, go across the row and find the appropriate column for the steel to wood connector, and finally find the fire resistance time (in minutes) for the given section:

Cover (mm)	High Temperature Epoxy	Timberlinx	Lagscrewbolt
10	3	6	3
20	17	19	17
30	31	33	31
40	44	47	45
50	58	61	59
60	72	75	73

70	86	89	86
80	100	103	100
90	114	117	114
100	128	131	128

Table 8-12 – Fire Resistance Table (min)

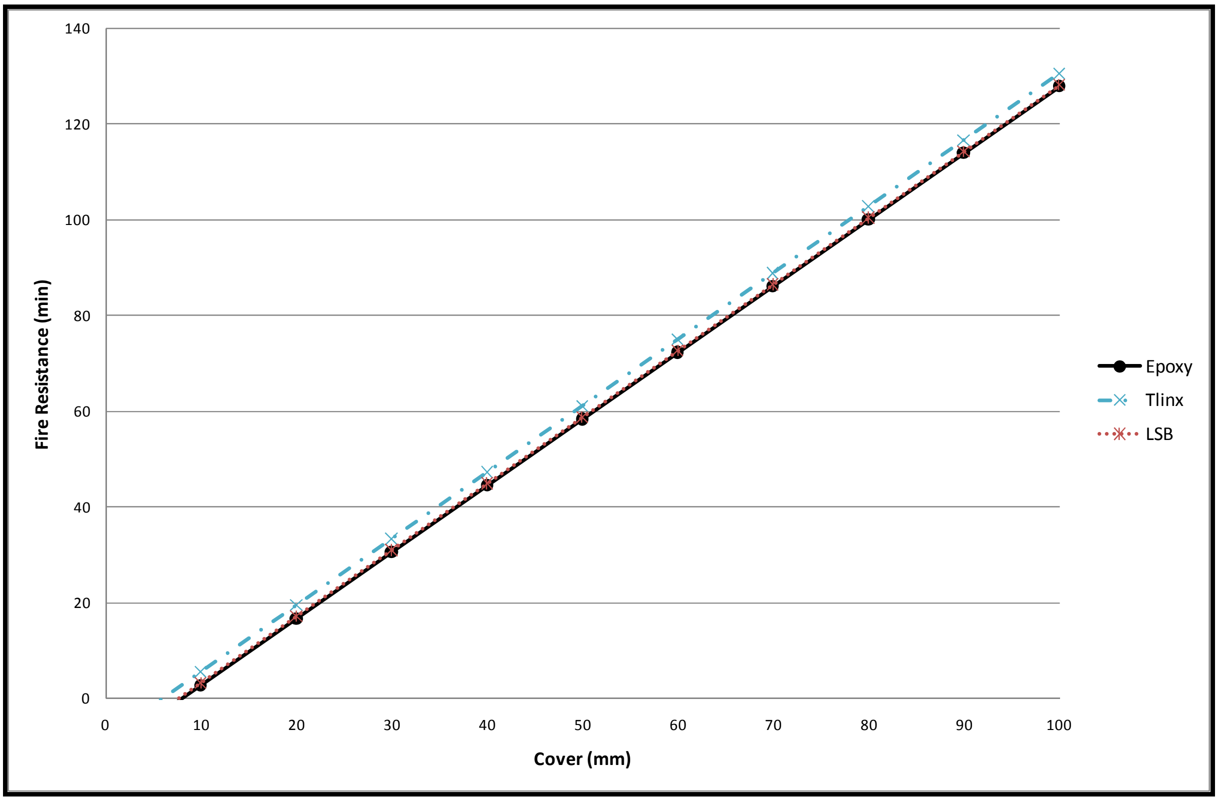

Figure 8-27 – Cover Design Plot

The plot presented in Figure 8-27 displays the same methodology as Table 8-12. The x-axis displays the cover depth while the y-axis shows the fire resistance. Determining the fire resistance for an existing connection requires one to enter the plot on the x-axis with the given cover depth, proceed to the designated curve for the type of steel to wood connector desired, and continue to the y-axis to determine the fire resistance for the connector.

8.9. Conclusions

Furnace testing was performed in the custom furnace to determine the fire behaviour and performance of steel to wood connections. The Fischer, JB Weld and West high temperature epoxies, as well as the Timberlinx and Lagscrewbolt products were subjected to the ISO 834 standard fire in the custom furnace. Results from furnace testing were used to generate equations for calculating the fire resistance of steel to wood connections.

Calculation methods rely on a limited number of furnace test samples for determining the fire resistance. Safety factors have not been included in the experimental equations, as current research is focused on a feasibility study as opposed to design implementation. Further testing and modification could evaluate the design and analysis equations and include a level of conservatism for structural design of steel to wood connections.

8.9.1. Epoxy Specimens

Furnace testing of high temperature epoxy specimens demonstrated an average fire resistance just greater than 20 minutes. Previous testing of all-purpose epoxy grouted steel threaded rod specimens (Harris, 2004) resulted in relatively low fire resistance times, around 12 minutes. Harris' experimentation also observed a number of epoxy adhesive failures. Furnace testing for the high temperature epoxies displayed significantly improved fire resistance as well as demonstrated wood failures as opposed to epoxy failures, suggesting improved strength for high temperature epoxy in fire conditions. Results suggest that connection fire resistance could be improved by increasing the cover depth protecting the epoxy grouted steel threaded rod.

Further testing could seek to engage in additional furnace tests for high temperature epoxy specimens. Future research could refine the established fire resistance equations for steel to wood connector use with high temperature epoxy.

8.9.2. Timberlinx Specimens

Furnace testing of Timberlinx connections displayed comparable fire resistance to high temperature epoxy specimens, with a fire resistance of 19 minutes. A modest body of knowledge was unable to provide any additional information regarding fire performance of Timberlinx connections for comparison. A failure mode analysis observed a brittle wood failure resulting from the reduced cross sectional area of LVL in line with the expanding anchor. Analysis suggests that additional cover could mitigate this failure mechanism and provide improved fire performance.

Future testing could evaluate the furnace test results and equations for calculating the Timberlinx fire resistance. Research using the Timberlinx product with different specimen sizes could clarify effects of cover depth on Timberlinx fire performance.

8.9.3. Lagscrewbolt Specimens

As an embedded steel to wood connector, it was surprising that the Lagscrewbolt product displayed the lowest fire resistance of the three types of experimental specimens. Furnace testing revealed a fire resistance of 17.6 minutes, nearly three minutes less than for the high temperature epoxy specimens. Analyzing the observed LVL confinement failure suggests that a greater amount of cover would increase the fire resistance for the connection.

Further experimentation with the Lagscrewbolt steel to wood connector could validate design and analysis equations. Varying the LVL specimen section and cover depth could evaluate the effect of heavy timber section properties on Lagscrewbolt connection fire performance and behaviour.

9. Discussion

The discussion section seeks to clarify issues concerning tensile testing, specimen selection, quality control, oven testing temperatures, applied loads for furnace testing, and finally, justifying experimental results as part of a feasibility analysis.

9.1. Tensile Testing

When engaging in experimentation of steel to wood connections, it is important to understand the intended purpose of the connection in use. In pre-stressed heavy timber buildings, epoxy grouted steel threaded rods and the Timberlinx and Lagscrewbolt proprietary mechanical fasteners can be used to provide a tensile connection between steel to wood elements. Should the elements go into compression, the compressive forces would be resolved in bearing by the heavy timber members.

The steel to wood connector is designed to resist tensile forces only, generally resulting from lateral loads within the building. Tensile testing was performed to enhance the understanding of steel to wood connections that are used to resist tensile forces in pre-stressed heavy timber structures.

9.2. Specimen Selection

Material and specimen selection was based on previous experimentation with epoxy grouted steel threaded rods in laminated veneer lumber (van Houtte, 2003 and Harris, 2004). Previous research engaged in tensile testing of all-purpose epoxy adhesive grout with high strength steel threaded rods for evaluating the steel to wood connection.

9.2.1. Laminated Veneer Lumber

A significant amount of laminated veneer lumber was required for experimentation. LVL was supplied by NelsonPine industries as the product was readily available in sufficient quantities at the appropriate specimen size and strength in a short amount of time.

9.2.2. Threaded Steel Rods

Before engaging in tensile testing, it was important to design the experimental specimens to force a connection failure at the steel to wood connection end. High strength steel threaded rods were selected to force connection failure at the experimental end by providing additional strength at the custom bracket end.

9.2.3. Epoxy Adhesives

Previous investigations evaluated the performance of all-purpose epoxy subjected to a variety of testing conditions. The introduction of high temperature epoxy suggested the potential for improved tensile strength at high temperatures, prompting additional research. Three high temperature epoxies were selected for tensile testing as part of the experimental regimen.

9.2.3.1. Fischer 'FIS V 360S' Injection Mortar

The Fischer 'FIS V 360S' Injection Mortar was selected for experimentation as it claimed to demonstrate considerable strength at high temperatures. Fischer product data claimed an F120 fire rating, making the Fischer high temperature epoxy adhesive a natural candidate for experimentation.

9.2.3.2. JB Weld 'Industro Weld'

The JB Weld 'Industro Weld' was selected for advertising an ability to maintain strength up to 260°C. This favourable temperature behaviour suggested that JB Weld high temperature epoxy be used in experimentation.

9.2.3.3. West System 'Z206' Epoxy Hardener and Adhesive Technologies 'ADR 310' Epoxy Resin

The West System 'Z206' Epoxy Hardener and Adhesive Technologies 'ADR 310' Epoxy Resin was selected due to previous testing utilizing the West brand of products (Harris, 2004).

Previous testing of West epoxies dealt with all-purpose epoxies, suggesting that high temperature epoxy may display improved behaviour with exposure to high temperatures.

9.2.4. Proprietary mechanical fasteners

Previous experimentation will limited to all-purpose epoxy adhesive grout for steel to wood connection testing. The recent introduction of the Timberlinx and Lagscrewbolt proprietary mechanical fasteners has offered alternatives to epoxy grouted solutions. Tensile testing of these proprietary mechanical fasteners provided guidance as to their performance and behaviour compared to traditional steel to wood connection techniques.

9.2.4.1. Timberlinx 'A475'

The Timberlinx 'A475' proprietary steel to wood connector provided a creative solution for the steel to wood connection. The Timberlinx steel to wood connector creates a mechanical connection with the laminated veneer lumber through the use of an expanding anchor bolt. This unique application prompted the experimentation of the Timberlinx product to further understand behaviour and performance, not only at ambient temperature, but also high temperatures and under fire conditions.

9.2.4.2. Lagscrewbolt

The Lagscrewbolt proprietary mechanical fastener acts as an embedded lag screw connection for connecting steel threaded rod into heavy timber. As an alternative product to epoxy grout, the Lagscrewbolt product was selected for tensile testing for two reasons. One, it presents an alternative to the epoxy grouted connection. Secondly, Lagscrewbolt uses a unique embedded connection that provides guidance for the fire resistance of embedded steel products.

9.3. Quality Control

Prior to tensile testing, it was important to establish tolerance limits for quality control of test specimens. All epoxy grouted and proprietary specimens were inspected to ensure that laminated veneer sizes were within tolerance limits, all drill holes were centrally located in test specimens, and steel threaded rods were arranged perpendicular to the face of the specimen. Testing conditions were administered as best as possible to maintain uniformity for all steel to wood test specimens.

During tensile testing, failed specimens were examined for acceptance of testing data. Aiming to analyze the ultimate tensile strength for steel to wood connections, any test specimens that failed at the fixed custom bracket end were retested to provide acceptable connection failures.

9.3.1. Fischer Epoxy Cold Tests

Following the completion of cold, oven, cooled and furnace tests, test specimen results were reviewed and evaluated to determine if any data points would require additional testing for validation and acceptance. One data point required additional testing: the Fischer epoxy cold test specimen. Five additional tests were performed with results presented in Table 9-1:

Test Specimen	Ultimate Load (kN)	Failure Mode
Fischer 01	51.0	1
Fischer 02	54.6	3
Fischer 03	61.2	1
Fischer 04	65.8	1
Fischer 05	61.2	3
Fischer 06	67.8	1

Fischer Average	60.3	1

Table 9-1 - Additional Fischer Epoxy Cold Test Results

An average of the six Fischer cold test samples has been used throughout the thesis. This value provides greater confidence based on a larger sample size. In addition, Failure Mode 1 has been accepted as the "typical" failure mode for Fischer epoxy tested at cold conditions. For reference, the furnace test load of 15kN was maintained for the Fischer furnace sample, as additional testing with the modified tensile load was not feasible.

One should also be aware that the Fischer cold test specimen is the only test data to represent more than one tensile test. Each of the remaining cold, oven, cooled and furnace test samples for the high-temperature epoxies and proprietary mechanical fasteners represents a single test sample. Further discussion of the single test issue can be found in Section 9.6.

9.4. Oven Testing

Oven testing was performed to enhance the understanding of steel to wood connection behaviour at high temperatures. Previous testing (Harris, 2004) performed heated tensile tests up to 100°C with the belief that epoxy grouted steel to wood connections do not retain any significant strength beyond this plateau. Current research aimed to experiment with steel to wood connections up to 300°C, the minimum charring temperature for laminated veneer lumber. However, only a 200°C maximum temperature was achieved for experimental oven testing.

Testing with laminated veneer lumber test specimens at 250°C was attempted in the University of Canterbury Civil Engineering Laboratory, with unfavourable results. While being left to heat overnight, test specimens began to smoulder and ignited, causing damage to the testing oven in addition to spreading smoke throughout the entire Civil Engineering Laboratory. This smoke spread incited serious health concerns from Civil Engineering Lab Technicians and also left a putrid smell throughout the Civil Engineering Laboratory for a significant amount of time.

While testing data for steel to wood connections tested at 250°C would provide additional information regarding specimen performance and behaviour, the maximum oven temperature was limited to 200°C to prevent further complications.

9.5. Furnace Testing – Applied Loads

Applied loads for the experimental furnace testing have been conservatively assumed to be approximately 30% of the ultimate capacity of each steel to wood connection type based on typical demand / capacity ratios for structural elements and on previous testing by Harris (2004). This is roughly equivalent to the load combination of 1.2DL + 0.5LL in the fire limit state. One should be aware, however, that the applied load may have a considerable effect on the fire resistance rating of the connection.

In terms of the experimental furnace testing, a constant tensile load greater than the assumed load would likely result in a reduced fire resistance time. Further, a constant applied load less than the assumed load would likely result in a greater fire resistance time.

To provide consistency in determining the applied loads for furnace testing, it is recommended that any future research with steel to wood connections using epoxy adhesives or mechanical fasteners utilise a constant applied load of approximately 30% of the ultimate capacity at ambient temperature for each connector.

9.6. Feasibility Analysis

The primary objective of experimentation was to gain a better understanding of steel to wood connection performance for specimens subjected to ambient, oven and cooled temperature conditions, as well as for fire conditions. Current testing has aimed to enhance the understanding of this connection type by performing tensile testing to temperatures exceeding previous limits with high strength epoxies as opposed to all-purpose epoxies, and proprietary mechanical fasteners as an alternative to epoxy use.

The current research engaged in a single test per specimen, per temperature, to attain a fundamental understanding of trends in steel to wood connector behaviour and performance. For example, experimental data for the West 50°C oven sample resulted from a single West epoxy grouted steel threaded rod specimen subjected to tensile testing at 50°C. Individual tests were performed as part of a feasibility study for steel to wood connections.

Assembling testing data for each specimen over a range of temperatures and testing conditions was intended to reveal trends and patterns in testing data. Analyzing the experimental data as a whole is intended to increase the fundamental understanding of these unique connection types. This comprehensive feasibility analysis is expected to provide guidance for where future research would be necessary and experimental results may be promising. Future testing with a greater number of tensile tests would increase the confidence in results and allow for quantitative analysis as opposed to qualitative analysis.

10. Conclusions and Recommendations

The sustainability and environmental advantages of pre-stressed heavy timber structures supports the need for further investigation of the fire risks associated with this innovative construction assembly. An extensive literature review was conducted to determine the fire resistance of pre-stressed heavy timber structural components. Laboratory testing was performed in the research project to determine the fire behaviour and performance of steel to wood connections. A summary of testing and test results are set forth below.

10.1. Summary of Testing

Experimental testing of steel to wood connections involved a four-phase procedure. Tensile testing with three high temperature epoxies and the Timberlinx and Lagscrewbolt proprietary mechanical fasteners provided ultimate strength values and failure modes that were used to evaluate the fire performance and behaviour of steel to wood connections.

The first three phases of experimentation involved tensile testing of steel to wood specimens in the Avery Testing Machine. Cold testing involved experimental specimens in tensile tests at ambient temperature conditions. Cold test results were used to establish a control ultimate strength and behaviour for ambient temperature performance.

Test specimens in oven tests were heated to temperatures ranging from 50°C to 100°C, at 50°C increments and subjected to tensile testing at high temperatures. Results from oven tests were used to establish the effects of increasing temperature on steel to wood connections.

The third phase of testing, cooled testing, proceeded similar to oven tests. Instead of testing at high temperatures, specimens were allowed to cool to ambient temperature prior to tensile testing. Results from cooled tests established the residual strength of steel to wood connections subjected to high temperature conditions and allowed to return to ambient conditions.

The final phase of testing, furnace testing, used the custom furnace to expose steel to wood connections to the ISO 834 standard fire. Test specimens were held at a constant tensile load and subjected to heating until failure. The time to failure was recorded and results were used to determine the fire performance and fire resistance of steel to wood connections under simulated fire conditions.

10.2. Summary of Test Results

Each of the four experimental phases were designed to evaluate the steel to wood connection fire performance and behaviour. Test results for the three high temperature epoxies and proprietary mechanical fasteners revealed trends and behaviours that were used for comparison with previous testing, and can be applied to future research.

Cold testing established a baseline ultimate strength and behaviour for test specimens. Results suggest that at cold conditions, epoxy grouted steel threaded rod connections provide greater strength than both the Timberlinx and Lagscrewbolt proprietary mechanical fasteners. Failure mechanisms at cold temperatures ranged from confinement failures to wood pull out failures and a tension parallel to grain failure.

Steel to wood connections were heated in the oven to determine the effect of increasing temperature on performance and behaviour. Ultimate strengths and failure mechanisms were recorded and compared to cold test results to the effects of exposure to high temperatures. Overall, ultimate strength decreased with increases in temperature. Results demonstrated a trend: the higher the temperature, the greater the strength loss in the experimental specimen. Interestingly, the failure mechanism for epoxy specimens transitioned from LVL failures at 50°C to epoxy adhesive failures at the epoxy-wood interface at 100°C and beyond.

Cooled tests involved heating epoxy grouted steel threaded rod specimens and allowing them to cool to ambient temperature prior to tensile testing. Cooled tests evaluated the residual strength of steel to wood connections following heating, simulating a minor fire event. Tests results demonstrated no significant difference in ultimate strength values when compared to cold strengths. Failure mechanisms remained constant for all test specimens, displaying confinement failures for specimens heated to 50°C and pull out failures for all other test specimens.

Furnace tests used the custom furnace to subject steel to wood test specimens to the ISO 834 standard fire. The time to failure was recorded and used to determine the fire resistance for each connection. Fire resistance values established the amount of time for which connections can be expected to meet structural performance requirements in a building fire.

Fire resistance times ranged between approximately 18 minutes and 23 minutes for steel to wood connections. The Lagscrewbolt product displayed the shortest fire resistance time at 17.6 minutes, while the Fischer high temperature epoxy demonstrated the longest time at 22.5 minutes. All failure mechanisms resulted in LVL confinement failures except for the Timberlinx product, which experienced tensile cracking of the wood at the expanding anchor. Surprisingly, furnace test specimens did not produce any epoxy failures. This suggests that the strength of the wood decreased at a faster rate than the strength of the high temperature epoxy.

10.3. Design Recommendations

When designing steel to wood connections, two primary options are available: the use of epoxy grouted steel threaded rods or proprietary mechanical fasteners. Traditional connections utilised all-purpose epoxy for the steel to wood connection in pre-stressed heavy timber buildings. Current research has provided guidance for alternatives to all-purpose epoxy, with experimentation of high temperature epoxy and the Timberlinx and Lagscrewbolt proprietary mechanical fasteners.

High temperature epoxy behaves similar to all-purpose epoxy with several improvements over traditional epoxy behaviour. High temperature epoxy maintained significantly greater strength when subjected to elevated temperatures, and also demonstrated considerably greater fire resistance for steel to wood connections.

The use of proprietary mechanical fasteners offers an alternative to epoxy grouted steel to wood connections. The Timberlinx and Lagscrewbolt products utilise a mechanical bond as opposed to an epoxy adhesive bond between the steel and wood materials. Despite lesser ultimate strength at ambient temperature compared to epoxy connections, proprietary mechanical fasteners display favourable performance at high temperatures.

While high temperature epoxy provides greater fire resistance than all-purpose epoxy for steel to wood connections, testing results suggest that the most effective method of providing additional fire resistance may be to increase the heavy timber cover depth. Additional cover depth provides greater insulation for preventing heat transfer to the core connection, as well as increased cover for the critical failure width for increasing the charring times and thus the fire resistance.

10.4. Future Research

Experimentation with steel to wood connections was performed to evaluate the performance and behaviour of steel to wood connections at cold, oven and fire conditions. Further research could engage in the following:

- Performing additional testing of steel to wood connections for cold, oven and cooled tests. Increasing the number of testing repetitions could provide a greater sample size for evaluating experimental results. This would allow for greater confidence in test data. Further research could provide quantitative equations for an ultimate strength analysis of steel to wood connections at ambient and high temperature conditions.
- Cooled strength values for two of the three high temperature epoxies maintained relatively equal ultimate strength values at cold and cooled conditions. Additional experimentation for cooled test specimens could evaluate the increased strength phenomena witnessed for the third epoxy. Further testing with the Fischer product and other high temperature epoxies could evaluate this unique behaviour.
- Furnace testing investigated the fundamental fire performance and fire resistance for steel to wood connections. Experimentation was limited to one specific cross section of laminated veneer lumber with a constant tensile load. Future furnace testing could vary the timber section size and cover depth to determine the effects of variable cross sectional area and cover depth on fire performance and fire resistance. Additional furnace testing could vary the tensile fire demand load during testing. Experimental testing with variable tensile fire demand could evaluate the effect of load ratios on steel to wood connector fire resistance.
- A failure mode analysis for experimental specimens displayed fundamental behaviour of steel to wood test specimens. Theoretically, it should be possible to calculate the failure loads for failure mechanisms occurring in laminated veneer lumber. Current testing with limited sample size was unable to establish trends in failure mechanism corresponding to the ultimate strength of the connection. Tensile testing with a greater number of specimens could clarify failure mechanism performance and behaviour.

10.5. Conclusions

The primary objective of this research report was to determine the fire resistance of pre-stressed heavy timber structures. An extensive literature review evaluated the fire resistance for pre-stressed heavy timber structural components. Analysis suggests that the connections are the weakest link in the structure. However, when connections are protected, analysis indicated that pre-stressed heavy timber construction provides inherent fire resistance of up to two hours with no additional fire protection for structural elements.

Secondary objectives involved evaluating the fire behaviour and performance of steel to wood connections at ambient, oven and simulated fire conditions. A four-phase series of experiments determined the fire behaviour and performance of high temperature epoxy and proprietary mechanical fasteners through cold, oven, cooled and furnace testing.

Experimentation with high temperature epoxy demonstrated significant strength at ambient temperature. Increases in temperature caused a reduction in ultimate strength in all test specimens. Testing demonstrated that ultimate strength could be recovered if heated specimens were allowed to cool to ambient temperature. Furnace testing of epoxy specimens provided equations for calculating the fire resistance of high temperature epoxy grouted steel threaded rod connections.

Proprietary mechanical fasteners provided alternative connection techniques to epoxy grout for steel to wood connections. Despite these products displaying lower ultimate strength at ambient temperature, proprietary mechanical fasteners demonstrated less strength loss at high temperatures. Ultimate strength values for proprietary mechanical fasteners experienced gradual declines with increases in temperature. Furnace testing of proprietary specimens provided methods for calculating the fire resistance of proprietary mechanical fasteners.

This research report has evaluated the fire resistance of pre-stressed heavy timber structures and evaluated the fire behaviour and performance of steel to wood connections. Ultimately, the fire performance of the heavy timber frame provides considerable inherent fire resistance for structural components. Additionally, designing heavy timber structures with a greater cover depth will provide additional fire protection and increase the fire resistance of structural components and steel to wood connections alike. Favourable fire performance for high temperature epoxies and proprietary mechanical fasteners suggests their use as part of the hybrid timber-energy dissipater connection in pre-stressed heavy timber structures is a feasible design solution.

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12. Appendix 1 – Typical Pre-Stressed Heavy Timber Beam Fire Resistance Calculation

12.1. Bending Analysis

Beam Size:

Width, b =	378	mm
Depth, d =	600	mm
Length, L =	8560	mm

Beam Gaps:

Width, b_{gap} =	126	mm
Depth, d_{gap} =	348	mm

Cold Conditions:

Design Load, $w_c = 1.2DL + 1.6LL$ =	60.8	kN/m
Bending Moment, $M^* = w_cL^2/8$ =	556.4	kNm
Section Modulus, Z_x =		
$Z_x = (bd^2/6) - (b_{gap}d_{gap}{}^2/6)$ =	21.2	$mm^3 \times 10^6$
Nominal Strength $M_n = k_df_bZ_x$ =	814	kNm
Design Strength, $M_n phi$ =	651	kNm

Fire Conditions:

Design Load, $w_f = 1.0DL+0.4LL$ =	4.70	kN/m
Bending Moment, $M^*_f = w_cL^2/8$ =	181	kN/m

Design Loads:

Dead Load, DL =	3.5	kPa
Live Load, LL =	3.0	kPa

Beam Span:

Span =	6750	mm

Distributed Loads:

DL x Span, w_{DL} =	23.6	kN/m
LL x Span, w_{LL} =	20.3	kN/m

Cold Factors:

[1]k_d =	0.80	
[2]phi =	0.80	
[3]f_b =	48.0	MPa

Fire Factors:

[1]k_d =	1.0	
[2]phi =	1.0	
[3]f_b =	48.0	MPa
[4]Fire burning rate, Beta =	0.72	mm/min
[5]Zero strength layer, d_{zero} =	7.0	mm
[6]k_{20} =	1.15	

Fire Resistance Analysis Equations:

Char depth, d_{char} =	elapsed time x Beta
Reduced width, b_{red} =	$b - 2d_{char}$
[7]Reduced depth, d_{red} =	$d - d_{char}$
Effective width, b_{eff} =	$b_{red} - 2d_{zero}$
[7]Effective depth, d_{eff} =	$d_{red} - d_{zero}$
Moment of Inertia, I_{red} =	$(b_{eff}d_{eff}{}^3) - (b_{gap}d_{gap}{}^3)/12$
Section modulus, Z_{red} =	$I_x / (d_{eff} / 2)$
Wood strength, M_f =	$k_dk_{20}f_bphi\ Z_{red}$

Notes:

[1]k_d = load duration factor from NZS 3603

[2]phi = φ strength reduction factor from NZS 3603

[3]f_b = characteristic bending strength from manufacturers data (see Table 4-1)

[4]beta = Laminated veneer lumber charring rate (Lane, 2001)

[5]d_{zero} = zero strength layer for the "Effective cross-section method" in Eurocode 5 (Buchanan, 2001)

[6]k_{20} = conversion factor for 5th to 20th characteristic percentile strength in Eurocode 5 (Buchanan, 2001)

[7]Effective depth, d_{eff} = reduction by (1)d_{zero} for 3 sided exposure

Time (min):	d_{char} (mm):	b_{red} (mm):	d_{red} (mm):	b_{eff} (mm):	d_{eff} (mm):	I_{red} (mm^4):	Z_{red} (mm^3x10^6):	M_f (kNm):	Result:
0	0.00	378	600	364	593	5.88E+09	19.8	1095	OK
10	7.20	364	593	350	586	5.41E+09	18.5	1020	OK
20	14.4	349	586	335	579	4.97E+09	17.2	948	OK
30	21.6	335	578	321	571	4.54E+09	15.9	878	OK
40	28.8	320	571	306	564	4.14E+09	14.7	811	OK
50	36.0	306	564	292	557	3.76E+09	13.5	746	OK
60	43.2	292	557	278	550	3.40E+09	12.4	683	OK
70	50.4	277	550	263	543	3.06E+09	11.3	623	OK
80	57.6	263	542	249	535	2.74E+09	10.2	565	OK
90	64.8	248	535	234	528	2.44E+09	9.22	509	OK
100	72.0	234	528	220	521	2.15E+09	8.25	456	OK
110	79.2	220	521	206	514	1.88E+09	7.32	404	OK
120	86.4	205	514	191	507	1.63E+09	6.43	355	OK
130	93.6	191	506	177	499	1.39E+09	5.58	308	OK
140	100.8	176	499	162	492	1.17E+09	4.76	263	OK
150	108.0	162	492	148	485	9.65E+08	3.98	220	OK
160	115.2	148	485	134	478	7.72E+08	3.23	178	NG

13. Appendix 2 – Typical Heavy Timber Column Fire Resistance Calculation

13.1. Axial Analysis

Column Size:

Width, b =	378	mm
Depth, d =	600	mm
Length, L =	3300	mm

Column Gaps:

Width, b_{gap} =	0	mm
Depth, d_{gap} =	0	mm

Cold Conditions:

Design Load, N_c = 1.2DL+1.6LL =	9.00	kPa
Axial Demand, N^* =		
$N^* = N_c Trib_{area}$ # Floors =	3463	kN
Cross Sectional Area, A = (bd) =	226800	mm^2
Nominal Strength, $N_n = k_d f_c A$ =	8165	kN
Design Strength, $N_n phi$ =	6532	kN

Fire Conditions:

Design Load, N_f = 1.0DL+0.4LL =	4.7	kPa
Axial Demand, N^*_f =		
$N^*_f = N_c Trib_{area}$ # Floors =	1808	kN

Design Loads:

Dead Load, DL =	3.50	kPa
Live Load, LL =	3.00	kPa

Tributary Area:

$Trib_{area}$ =	64.1	m^2

Floors:

# Floors =	6	

Cold Factors:

[1]k_d =	0.80	
[2]phi =	0.80	
[3]f_c =	45.0	MPa

Fire Factors:

[1]k_d =	1.0	
[2]phi =	1.0	
[3]f_c =	45.0	MPa
[4]Fire burning rate, Beta =	0.72	mm/min
[5]Zero strength layer, d_{zero} =	7.0	mm
[6]k_{20} =	1.15	

Fire Resistance Analysis Equations:

Char depth, d_{char} =	elapsed time x Beta	
Reduced width, b_{red} =	$b - 2d_{char}$	(for exposure on 4 sides)
Reduced depth, d_{red} =	$d - 2d_{char}$	(for exposure on 4 sides)
Effective width, b_{eff} =	$b - b_{red} - 2d_{zero}$	(for exposure on 4 sides)
Effective depth, d_{eff} =	$d - d_{red} - 2d_{zero}$	(for exposure on 4 sides)
Cross Sectional Area, A_{red} =	$b_{red} d_{red}$	
Wood Strength, N_f =	$k_d k_{20} f_c phi\ A_{red}$	

Notes:

[1]k_d = load duration factor from NZS 3603

[2]phi = φ strength reduction factor from NZS 3603

[3]f_b = characteristic bending strength from manufacturers data (see Table 4-1)

[4]beta = Laminated veneer lumber charring rate (Lane, 2001)

[5]d_{zero} = zero strength layer for the "Effective cross-section method" in Eurocode 5 (Buchanan, 2001)

[6]k_{20} = conversion factor for 5th to 20th characteristic percentile strength in Eurocode 5 (Buchanan, 2001)

Time (min):	d_{char} (mm):	b_{red} (mm):	d_{red} (mm):	b_{eff} (mm):	d_{eff} (mm):	A_{red} (mm^2):	N_f (kN):	Result:
0	0.00	378	600	364	586	2.13E+05	11038	OK
10	7.20	364	586	350	572	2.00E+05	10341	OK
20	14.4	349	571	335	557	1.87E+05	9666	OK
30	21.6	335	557	321	543	1.74E+05	9011	OK
40	28.8	320	542	306	528	1.62E+05	8378	OK
50	36.0	306	528	292	514	1.50E+05	7767	OK
60	43.2	292	514	278	500	1.39E+05	7177	OK
70	50.4	277	499	263	485	1.28E+05	6609	OK
80	57.6	263	485	249	471	1.17E+05	6062	OK
90	64.8	248	470	234	456	1.07E+05	5536	OK
100	72.0	234	456	220	442	9.72E+04	5032	OK
110	79.2	220	442	206	428	8.79E+04	4550	OK
120	86.4	205	427	191	413	7.90E+04	4088	OK
130	93.6	191	413	177	399	7.05E+04	3649	OK
140	100.8	176	398	162	384	6.24E+04	3231	OK
150	108.0	162	384	148	370	5.48E+04	2834	OK
160	115.2	148	370	134	356	4.75E+04	2459	OK
170	122.4	133	355	119	341	4.07E+04	2105	OK
179	128.9	120	342	106	328	3.49E+04	1805	NG

13.2. Column Buckling Analysis

Column Size:

Width, b =	378	mm
Depth, d =	600	mm
Length, L =	3300	mm

Column Gaps:

Width, b_{gap} =	0	mm
Depth, d_{gap} =	0	mm

Slenderness:

Slenderness <	200	
Bracing, k =	1.0	
Unbraced Length, l =	3400	mm

Tributary Area:

$Trib_{area}$ =	64.1	m^2

Floors:

# Floors =	6

Cold Factors:

[1]k_d =	0.80	
[2]phi =	0.80	
[3]f_c =	45.0	MPa

Fire Factors:

[1]k_d =	1.0
[2]phi =	1.0

Fire Resistance Analysis Equations:

Char depth, d_{char} =	elapsed time x Beta	[3]f_c =	45.0	MPa
Reduced width, b_{red} =	$b - 2d_{char}$	[4]Fire burning rate, Beta =	0.72	mm/min
Reduced depth, d_{red} =	$d - 2d_{char}$	[5]Zero strength layer, d_{zero} =	7.0	mm
Effective width, b_{eff} =	$b - b_{red} - 2d_{zero}$	[6]k_{20} =	1.15	
Effective depth, d_{eff} =	$d - d_{red} - 2d_{zero}$			
Moment of Inertia, I_{red} =	$(b_{eff}d_{eff}^3) - (b_{gap}d_{gap}^3)/12$			
Cross Sectional Area, A_{red} =	$b_{red}d_{red}$			
Radius of Gyration, r_{red} =	$\text{sqrt}((d_{eff}b_{eff}^3)/(b_{red}d_{red})$			
Slenderness Ratio =	kl/r_{red}			

Notes:

[1]k_d = load duration factor from NZS 3603

[2]phi = φ strength reduction factor from NZS 3603

[3]f_b = characteristic bending strength from manufacturers data (see Table 4-1)

[4]beta = Laminated veneer lumber charring rate (Lane, 2001)

[5]d_{zero} = zero strength layer for the "Effective cross-section method" in Eurocode 5 (Buchanan, 2001)

[6]k_{20} = conversion factor for 5th to 20th characteristic percentile strength in Eurocode 5 (Buchanan, 2001)

Time (min):	d_{char} (mm):	b_{red} (mm):	d_{red} (mm):	b_{eff} (mm):	d_{eff} (mm):	I_{red} (mm^4):	A_{red} (mm^2):	r_{red} (mm)	Slenderness Ratio	Result:
0	0.0	378	600	364	586	2.36E+09	2.13E+05	105	32.4	OK
10	7.2	364	586	350	572	2.04E+09	2.00E+05	101	33.7	OK
20	14.4	349	571	335	557	1.75E+09	1.87E+05	96.8	35.1	OK
30	21.6	335	557	321	543	1.49E+09	1.74E+05	92.6	36.7	OK
40	28.8	320	542	306	528	1.27E+09	1.62E+05	88.5	38.4	OK
50	36.0	306	528	292	514	1.07E+09	1.50E+05	84.3	40.3	OK
60	43.2	292	514	278	500	8.91E+08	1.39E+05	80.1	42.4	OK
70	50.4	277	499	263	485	7.37E+08	1.28E+05	76.0	44.7	OK
80	57.6	263	485	249	471	6.04E+08	1.17E+05	71.8	47.3	OK
90	64.8	248	470	234	456	4.90E+08	1.07E+05	67.7	50.2	OK
100	72.0	234	456	220	442	3.92E+08	9.72E+04	63.5	53.5	OK
110	79.2	220	442	206	428	3.10E+08	8.79E+04	59.4	57.3	OK
120	86.4	205	427	191	413	2.41E+08	7.90E+04	55.2	61.6	OK
130	93.6	191	413	177	399	1.84E+08	7.05E+04	51.0	66.6	OK
140	101	176	398	162	384	1.37E+08	6.24E+04	46.9	72.5	OK
150	108	162	384	148	370	1.00E+08	5.48E+04	42.7	79.6	OK
160	115	148	370	134	356	7.07E+07	4.75E+04	38.6	88.2	OK
170	122	133	355	119	341	4.82E+07	4.07E+04	34.4	98.8	OK
180	130	119	341	105	327	3.13E+07	3.42E+04	30.3	112	OK

190	137	104	326	90.4	312	1.92E+07	2.82E+04	26.1	130	OK
200	144	90.0	312	76.0	298	1.09E+07	2.26E+04	21.9	155	OK
210	151	75.6	298	61.6	284	5.52E+06	1.75E+04	17.8	191	OK
212	153	72.7	295	58.7	281	4.74E+06	1.65E+04	17.0	200	NG

14. Appendix 3 – Typical Pre-Stressed Heavy Timber Wall Fire Resistance Calculation

14.1. Axial Analysis

Wall Size:

Width, b =	1000	mm
Depth, d =	252	mm
Length, L =	4000	mm

Wall Gaps:

Width, b_{gap} =	0	mm
Depth, d_{gap} =	0	mm

Cold Conditions:

Design Load, $N_c = 1.2DL+1.6LL$ =	9.00	kPa
Axial Demand, N^* =		
$N^* = N_c Trib_{area}$ # Floors =	243	kN
Cross Sectional Area, $A = (bd)$ =	252000	mm^2
Nominal Strength, $N_n = k_d f_c A$ =	9072	kN
Design Strength, N_n phi =	7258	kN

Fire Conditions:

Design Load, $N_f = 1.0DL + 0.4LL$ =	4.70	kN
Axial Demand, N^*_f =		
$N^*_f = N_c Trib_{area}$ # Floors =	127	kN

Design Loads:

Dead Load, DL =	3.50	kPa
Live Load, LL =	3.00	kPa

Tributary Area:

$Trib_{area}$ =	4.50	m^2

Floors:

# Floors =	6	

Cold Factors:

[1] k_d =	0.80	
[2] phi =	0.80	
[3] f_c =	45.0	MPa

Fire Factors:

[1] k_d =	1.0	
[2] phi =	1.0	
[3] f_c =	45.0	MPa
[4] Fire burning rate, Beta =	0.72	mm/min
[5] Zero strength layer, d_{zero} =	7.0	mm
[6] k_{20} =	1.15	

Fire Resistance Analysis Equations:

Char depth, d_{char} =	elapsed time x Beta
Reduced width, b_{red} =	$b - 2d_{char}$
Reduced depth, d_{red} =	$d - 2d_{char}$
Effective width, b_{eff} =	$b - b_{red} - 2d_{zero}$
Effective depth, d_{eff} =	$d - d_{red} - 2d_{zero}$
Moment of Inertia, I_{red} =	$(b_{eff}d_{eff}^3) - (b_{gap}d_{gap}^3)/12$
Cross Sectional Area, A_{red} =	$b_{red}d_{red}$
Radius of Gyration, r_{red} =	$\mathrm{sqrt}((d_{eff}b_{eff}^3)/(b_{red}d_{red})$
Slenderness Ratio =	kl/r_{red}

Notes:

[1] k_d = load duration factor from NZS 3603

[2]phi = φ strength reduction factor from NZS 3603

[3]f_b = characteristic bending strength from manufacturers data (see Table 4-1)

[4]beta = Laminated veneer lumber charring rate (Lane, 2001)

[5]d_{zero} = zero strength layer for the "Effective cross-section method" in Eurocode 5 (Buchanan, 2001)

[6]k_{20} = conversion factor for 5th to 20th characteristic percentile strength in Eurocode 5 (Buchanan, 2001)

Time (min):	d_{char} (mm):	b_{red} (mm):	d_{red} (mm):	b_{eff} (mm):	d_{eff} (mm):	A_{red} (mm^2):	N_f (kN):	Result:
0	0.00	1000	252	986	238	2.35E+05	12144	OK
10	7.20	986	238	972	224	2.17E+05	11243	OK
20	14.4	971	223	957	209	2.00E+05	10363	OK
30	21.6	957	209	943	195	1.84E+05	9504	OK
40	28.8	942	194	928	180	1.67E+05	8667	OK
50	36.0	928	180	914	166	1.52E+05	7852	OK
60	43.2	914	166	900	152	1.36E+05	7058	OK
70	50.4	899	151	885	137	1.21E+05	6285	OK
80	57.6	885	137	871	123	1.07E+05	5534	OK
90	64.8	870	122	856	108	9.28E+04	4804	OK
100	72.0	856	108	842	94.0	7.91E+04	4096	OK
110	79.2	842	94	828	79.6	6.59E+04	3409	OK
120	86.4	827	79	813	65.2	5.30E+04	2744	OK
130	93.6	813	65	799	50.8	4.06E+04	2100	OK
140	101	798	50	784	36.4	2.86E+04	1478	OK
150	108	784	36	770	22.0	1.69E+04	877	OK
160	115	770	22	756	7.60	5.74E+03	297	OK
164	118	764	16	750	1.84	1.38E+03	71.4	NG